APPENDIX AND REFERENCES

POTENTIAL FAILURE MODE ANALYSIS
KOOTENAI DEVELOPMENT IMPOUNDMENT DAM

LINCOLN COUNTY, MONTANA

August 23, 2011

Prepared for:

The Remedium Group

Prepared By:

Billmayer & Hafferman, Inc.
Kalispell, MT

Appendix 1

LRC-06 Flume

LRC-06 Outlet

LRC-06 Inlet

Carney Creek Flow above Flume

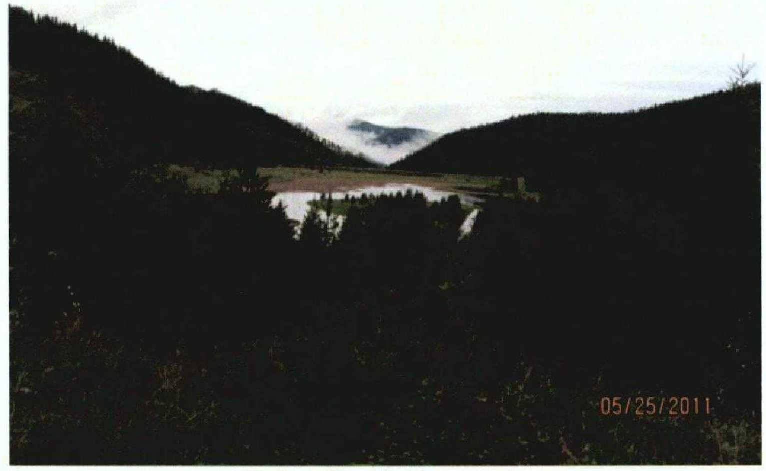
CC-02 Flume

CC-02 Outlet

CC-02 Inlet

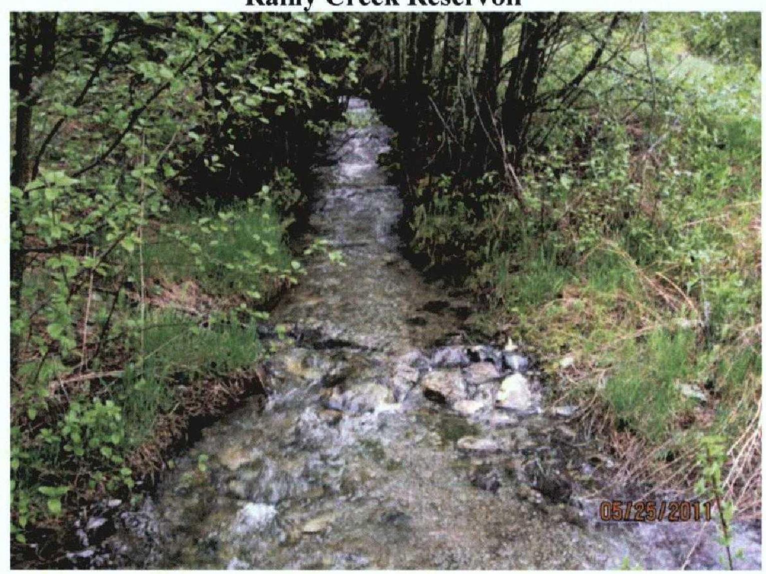
LRC-02 Flume

LRC-02 Inlet



Rainy Creek Reservoir

LRC-02 Outlet



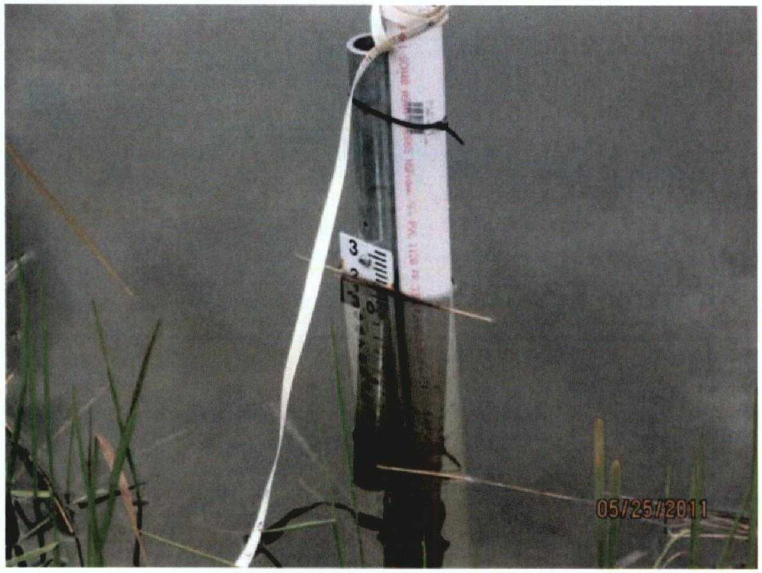
Flows above Fleetwood Creek Flume

Fleetwood Creek Flume

URC-02 Flume

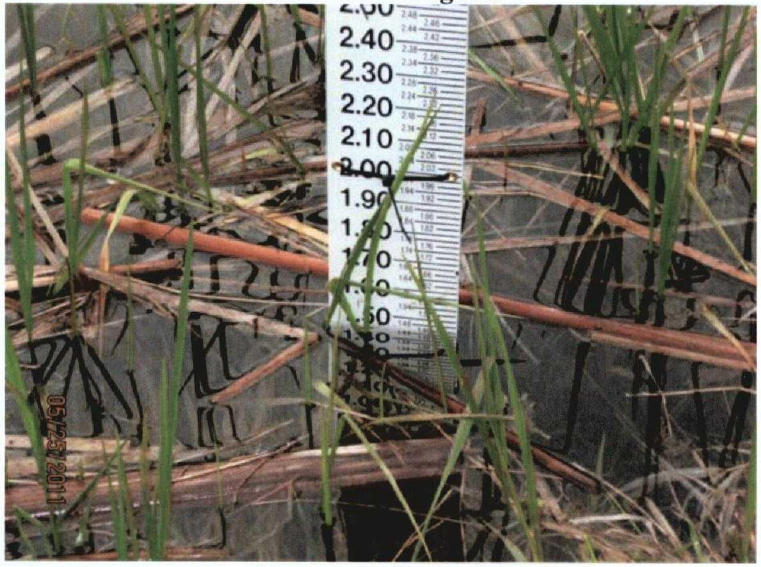
Fleetwood Creek Outlet

URC-02 Inlet



Reservoir Gauge #1

Steep Slope above Reservoir



Reservoir Gauge #2

Upstream Embankment from Staff Gauge

2191 Third Ave. East • P.O. Box 1139 Kalispell, MT 59903-1139 Phone: 406-257-8708 Fax: 406-257-8710

Reservoir Inlets from Staff Gauge

Principal Spillway Trash Rack

Upstream Toe of Embankment from Road

Principal Spillway Box Culvert Entrance

Box Culvert Inlet

Spillway Channel above Box Culvert

Box Culvert

Upstream Face from P5

Upstream Crest from P5

Downstream Crest from Left Abutment

Spillway Chute below Box Culvert

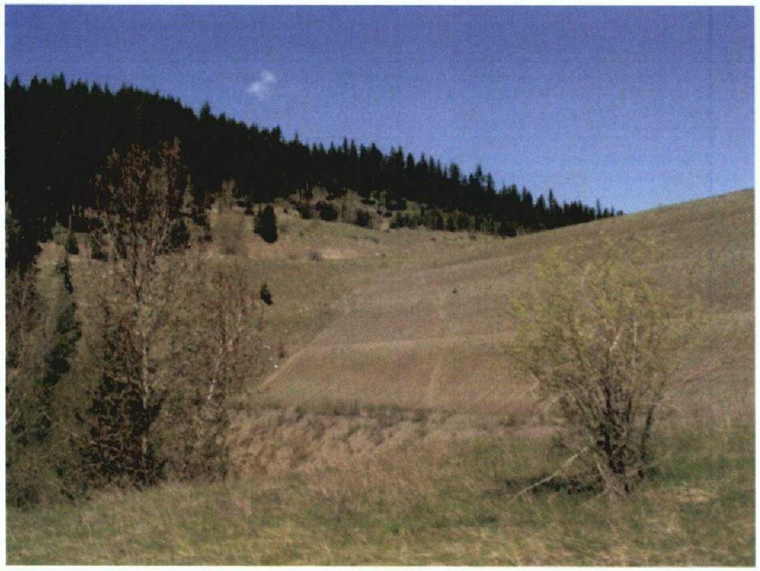
Downstream Right Abutment

Downstream Face of Dam

Looking across Downstream Embankment to Right Abutment

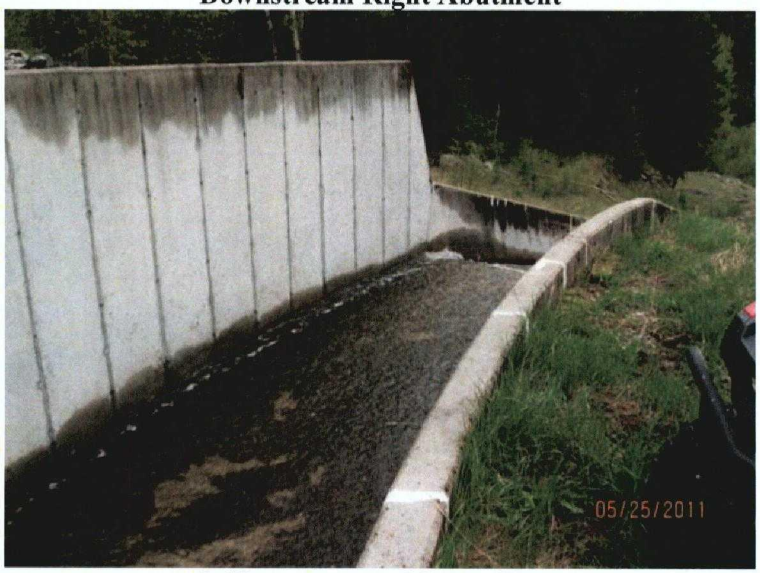
Downstream Left Abutment

Downstream Face of Embankment



Downstream Right Abutment

Steep Chute



Spillway at top of Steep Chute

Pooling Water at Downstream Left Toe

Drain 1 and Drain 2

Drain 1 Flow

Inside Drain 1

Inside Drain 2

Drain 3

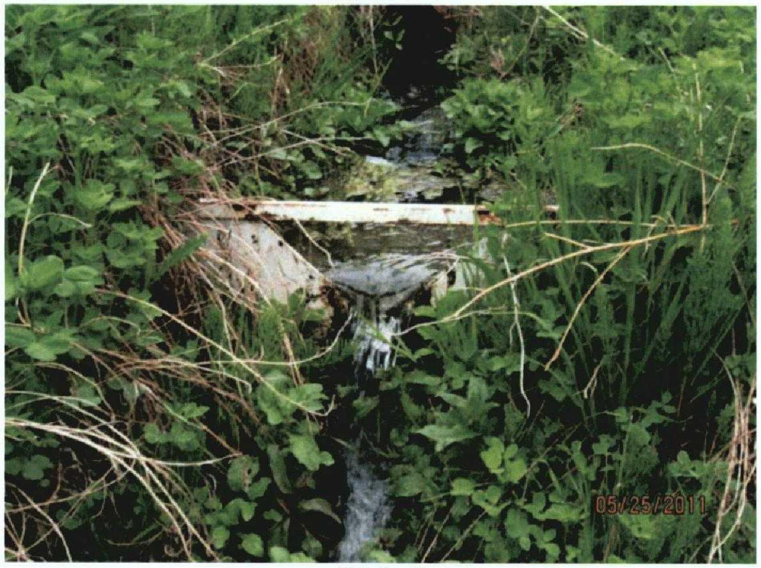
Drain 4

Inside Drain 3

Inside Drain 4

Flume 1-4

Drain 5



Weir 5

Inside Drain 5

Drain 6

Inside Drain 6

Drain 6 Flow

Seepage near Drain 7

Drain 7

Drain 8

Inside Drain 7

Inside Drain 8

Flume 7-8

Decant Line Outlet in Embankment

Seepage near Drain 8

Seepage Near Drain 9

Drain 9

Seepage between Drain 9 and Drain 13

Inside Drain 9

Inside Drain 13

Seepage between Drain 13 and Drain 10

Inside Drain 10

Drain 10 and Drain 11

Inside Drain 11

Weir 12

Drain 12 Flow

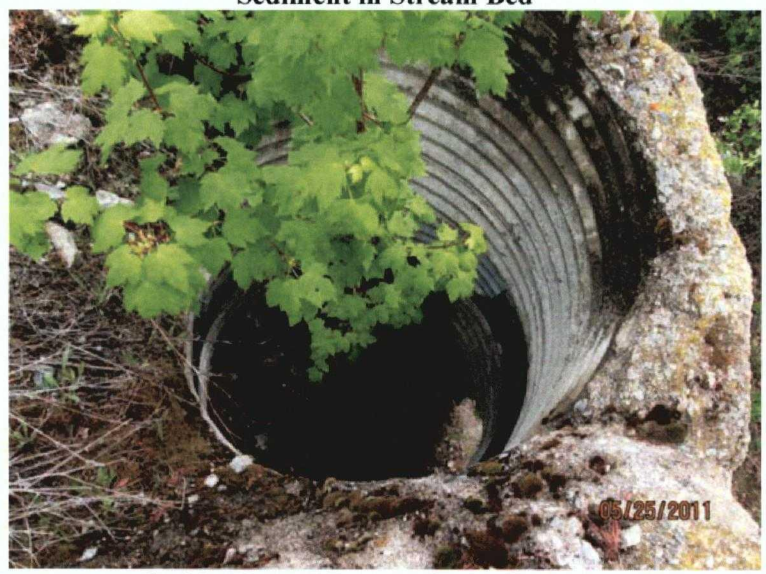
Drain 12

Seepage into Drain 12 above Weir

Saturated Area above Drain 12

Sediment in Stream Bed

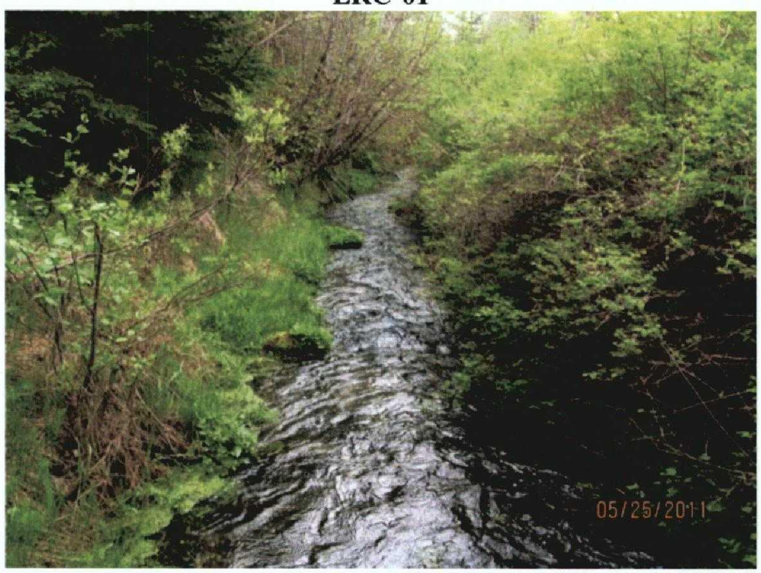
Water Seepage near Drain 12



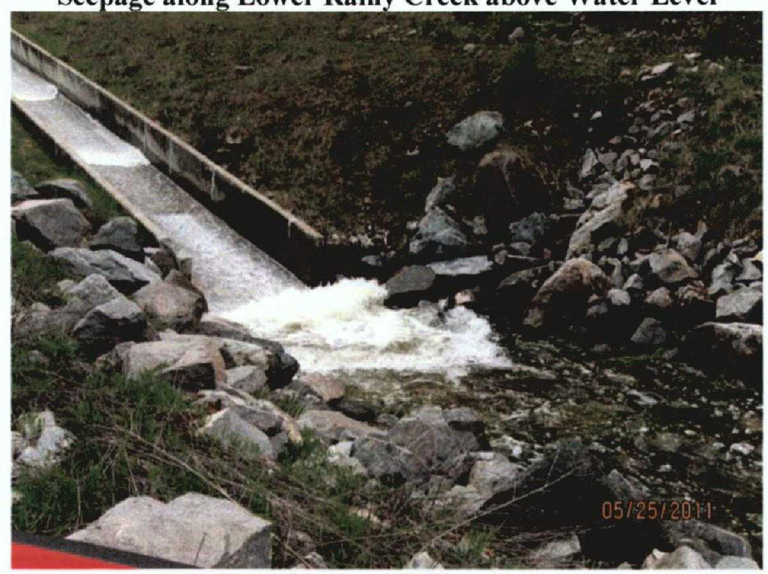
Inside Phase 5 Decant Outlet Structure

LRC-01

Seepage along Lower Rainy Creek above Water Level



Lower Rainy Creek below LRC-01



Steep Chute at Stilling Basin

Riprap in Stilling Basin

Trash Rack on Mill Pond

F-Seep Flume

Mill Pond Spillway

Mill Pond

Mill Pond Embankment Crest to Right

Mill Pond Embankment Crest to Left

Appendix 2

TARGET SHEET

EPA REGION VIII

SUPERFUND DOCUMENT MANAGEMENT SYSTEM

DOCUMENT NUMBER: 1216491

SITE NAME: LIBBY ASBESTOS SITE

DOCUMENT DATE: 08/23/2011

DOCUMENT NOT SCANNED

Due to one of the following reasons:

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- ☐ 3-DIMENSIONAL
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- ☐ POOR LEGIBILITY
- ☐ OTHER
- ☐ NOT AVAILABLE
- ☐ TYPES OF DOCUMENTS NOT TO BE SCANNED (Data Packages, Data Validation, Sampling Data, CBI, Chain of Custody)

DOCUMENT DESCRIPTION:

2 DVDs -
KDID Toe Drain Inspection Drain 1 & Drain 2, March 1, 2, 3, 2010 Volume 1
KDID Toe Drain Inspection II Drain 3, Drain 2 & Drain 6, May 21, 2010
Volume 1

Contact the Superfund Records Center to view available document.
(303) 312-6473

TARGET SHEET

EPA REGION VIII

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☐ OTHER

☐ NOT AVAILABLE

☐ TYPES OF DOCUMENTS NOT TO BE SCANNED
(Data Packages, Data Validation, Sampling Data, CBI, Chain of Custody)

DOCUMENT DESCRIPTION:

3 DVDs -
KDID Toe Drain Inspection Drains 11, 10, 9, 8 & 6, March 1, 2, 3, 2010 Volume 2
KDID Toe Drain Inspection Drains 3, 4, 5 & 12, March 1, 2, 3, 2010 Volume 3
KDID Toe Drain Inspection All Drains Video Condensed March 1, 2, 3, 2010
Volume 4

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TARGET SHEET

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DOCUMENT NUMBER: 1216491

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- ☐ NOT AVAILABLE
- ☐ TYPES OF DOCUMENTS NOT TO BE SCANNED (Data Packages, Data Validation, Sampling Data, CBI, Chain of Custody)

DOCUMENT DESCRIPTION:

2 DVDs -
KDID PFMA Meeting Video 5/26/2011
KDID Updated PFMA Files with Reference File Locator 5/13/2011

Contact the Superfund Records Center to view available document.
(303) 312-6473

Appendix 3

A3-Reference 1

Kootney

15

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

RAINY CREEK BASIN
ZONOLITE TAILINGS DAM

LIBBY, MONTANA
MT-1470

PREPARED FOR:

THE HONORABLE TED SCHWINDEN
GOVERNOR OF THE STATE OF MONTANA

W.R. GRACE & COMPANY
CONSTRUCTION PRODUCTS DIVISION
(OWNER AND OPERATOR)

PREPARED BY:

MORRISON - MAIERLE, INC.

CONSULTING ENGINEERS

SEPTEMBER 1981

U.S. ARMY
CORPS OF ENGINEERS
Seattle
DISTRICT

PHASE I INSPECTION REPORT
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SEPTEMBER 1981

CONTENTS

Paragraph		Page
	EXECUTIVE SUMMARY	iv
	PERTINENT DATA	vii
	CHAPTER 1 - BACKGROUND	
1.1	INTRODUCTION	1
1.1.1.	Authority and Scope	1
1.1.2	Purpose	2
1.1.3	Inspection	2
1.2	DESCRIPTION OF PROJECT	3
1.2.1	General	3
1.2.2	Regional Geology and Seismicity	4
1.2.3	Site Geology	4
1.2.4	Design and Construction History	5
	CHAPTER 2 - INSPECTION AND RECORDS EVALUTION	
2.1	HYDRAULICS AND STRUCTURES	7
2.1.1	Spillway	7
2.1.2	Outlet/Decant Tower	8
2.1.3	Rainy Creek Diversion	8
2.1.4	Freeboard	8
2.2	HYDROLOGY, CLIMATOLOGY, AND PHYSIOGRAPHY	9
2.2.1	General	9
2.2.2.	Reservoir Storage	10
2.2.3	Estimated Probable Maximum Flood	10
2.2.4	Flood Routing	11
2.3	GEOTECHNICAL EVALUATION	11
2.3.1.	Dam	11
2.3.2	Foundation Conditions, Seepage, and Drainage	13
2.3.3	Stability	16
2.4	PROJECT OPERATION AND MAINTENANCE	17
2.4.1.	Dam	17
2.4.2.	Reservoir	17
2.4.3.	Warning Plan	17

CONTENTS - Continued

Paragraph		Page
	CHAPTER 3 - FINDINGS AND RECOMMENDATIONS	
3.1	FINDINGS	18
3.1.1	Size, Hazard Classification, and Safety Evaluation	18
3.1.2	Embankment	18
3.1.3.	Spillway and Reservoir Capacity	19
3.1.4	Outlet Works/Decant Tower	19
3.1.5	Operation and Maintenance	19
3.2	RECOMMENDATIONS	20
	REFERENCES	21
	APPENDIX	
	Correspondence	

LIST OF PLATES

Plate 1 Vicinity Map
Plate 2 Zonolite Tailings Dam Area Map
Plate 3 Zonolite Tailings Dam Site Plan Phase 5
Plate 4 Zonolite Tailings Dam - Plan View
Plate 5 Zonolite Tailings Oam Phase 5 Addition
Plate 6 Geology Map
Plate 7 Zonolite Tailings Dam Spillway Design Phase 5 Typical Sections
Plate 8 Zonolite Tailings Dam Combined Decant Line and Spillway Rating Curve

LIST OF PHOTOS

Photo 1 Aerial View of Zonolite Tailings Dam Looking Upstream

Photo 2 Aerial View of Zonolite Dam and Area Immediately Downstream (7-25-80)

Photo 3 Phase 5 - Decant Tower (7-81)

Photo 4 Rainy Creek Diversion Structure
Foreground: Entrance to Rainy Creek -
Background: Entrance to Diversion Pipeline
(8-13-81)

Photo 5 Rainy Creek Diversion Pipeline Along Rainy Creek Road (7-81)

Photo 6 Zonolite Spillway and Downstream Face (8-13-81)

Photo 7 Spillway Chute and Approach Channel (8-13-81)

Photo 8 Spillway Approach Channel Looking Downstream (8-13-81)

Photo 9 Spillway Chute Looking Upstream Near Crest. Note Overlapping Joints (7-81)

Photo 10 Spillway Foundation Collar (8-13-81)

Photo 11 Erosion of Backfill and Undermining of Spillway Chute (7-81)

Photo 12 Bends in Spillway Chute Looking Downstream (7-81)

Photo 13 Downstream End of Spillway Chute Adjacent to Haul Road (8-13-81)

Photo 14 East Abutment Drain for Benches on Downstream Face (8-13-81)

Photo 15 West Abutment Area. Note Rill Erosion (8-13-81)

Photo 16 Downstream Face Looking Towards the West (8-13-81)

Photo 17 Upstream Face Looking Towards the West (8-13-81)

Photo 18 Zonolite Tailings Dam - Phase 5 - Crest (8-13-81)

Photo 19 Foundation and Spring Drains. Note Wetted Front (8-13-81)

Photo 20 First Exposed Drain Closest to East (left) Abutment (8-13-81)

Photo 21 Seepage from Bedding Material Around Spring Drain (8-13-81)

Photo 22 Foundation and Spring Drains Looking Towards the East (8-13-81)

Photo 23 Third Exposed Foundation Drain from East Abutment (8-13-81)

Photo 24 Fourth Exposed Foundation Drain from East Abutment (8-13-81)

Photo 25 Fifth Exposed Foundation Drain from East Abutment (8-13-81)

EXECUTIVE SUMMARY

Under contract with the State of Montana Department of Natural Resources and Conservation and with representation from Department of Natural Resources and Conservation and the Construction Products Division of W.R. Grace and Company, Morrison-Maierle, Inc. inspected Zonolite Tailings Dam on 25 July 1980 under the authority of Public Law 92-367. At that time the project was undergoing Phase 5 construction to raise the dam 35 feet. A followup inspection was conducted on 13 August 1981 upon construction completion. The dam is located in Lincoln County about 6.5 miles northeast of Libby, Montana in the Rainy Creek Basin.

This report was compiled from information obtained during onsite inspections, review of the construction log and plans, and analysis of available information. Findings were compared with engineering criteria that are currently accepted by most private and public agencies engaged in dam design, construction and operation.

FINDINGS AND EVALUATION

Zonolite Tailings Dam is owned and operated by W.R. Grace and Company and is located on private land. The reservoir is used to contain mine tailings. As tailings are deposited, the dam is raised in phases to provide storage. The dam, designed by Bovay Engineers, Inc. of Spokane, Washington, and Harding-Lawson Associates of Novate, California, will eventually be 200-feet high. Presently, it is 135-feet high (crest elevation 2,925 feet NGVD). The reservoir normally stores only enough water to accomplish tailings settling. A 48-inch diameter corrugated metal pipe intercepts Rainy Creek above the 3,000 foot elevation and diverts flow downstream of the dam. Existing manual controls at the diversion structure do not prevent storm flows from entering the reservoir. This report evaluates the 135-foot high dam assuming a tailings elevation of 2,873.0 feet NGVD (July 1980 inspection) and a dam crest elevation of 2,925 feet NGVD. Under these conditions, the project is capable of impounding about 2,120 acre-feet of water at spillway crest elevation 2,920 feet NGVD and 2,450 acre-feet at dam crest elevation 2,925 feet NGVD. All elevations used in this report are based on owner-supplied design drawings which correspond to the National Geodetic Vertical Datum (NGVD).

On the basis of criteria in U.S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Reference 1), the project is large in size. The dam is located such that its failure could endanger more than a few lives and cause excessive economic loss. However, no dam breach analysis or routing of a dam breach flood was made for the downstream area. The conclusions on probable damage are based on brief field visits and engineering judgement.

The project is classified as having a high (Category 1) downstream hazard potential. Inspection criteria (Reference 1) recommend that a large size project with a high downstream hazard potential be capable of safely handling the probable maximum flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region.

An estimated thunderstorm PMF was developed for the 9.7-square mile drainage basin during this dam safety study. The PMF resulting from the 6-hour thunderstorm has an estimated volume of 3770 acre-feet and a peak flow of 43,400 cfs The spillway has a maximum discharge capacity of 200 cfs with the reservoir at assumed top of dam, elevation 2,925 feet. The decant tower with minimum inlet elevation at 2890.1 feet has a maximum discharge capacity of 18 cfs and is used to maintain a steady level on the settling pond so that tailings will settle out and only clear water is discharged downstream. The routing of the PMF was started with the reservoir four feet above the inlet elevation of the decant line, which is the anticipated reservoir elevation prior to the occurrence of a flood of this magnitude. Routing of the PMF indicates that the dam is overtopped when approximately 55 percent of the total flood volume enters the reservoir. A flood with a hydrograph having ordinates corresponding to 45 percent of the PMF hydrograph ordinates is just controlled by the project. Larger floods would overtop the dam. These percentages of the PMF are valid only for conditions immediately after the completion of Phase 5. As tailings are deposited, there will be less storage available for floodwaters and the dam will overtop at PMF percentages smaller than those indicated above. The dam is constructed of materials that could quickly erode and rapidly fail if overtopped by floodwaters. Such failure could endanger lives immediately downstream at the screen plant and cause extensive damage to the highway, property and buildings. Because the project cannot safely handle the recommended spillway design flood (SDF), which is the full PMF, Zonolite Tailings Dam does not conform with inspection guideline hydraulic and hydrologic criteria.

During construction of the various phases of the dam, Bovay Engineering (for the starter dam only) and Harding-Lawson provided engineering services that included observation, consultation, and material testing. Visual inspection of the dam revealed no evidence of cracking, settlement or slope instability. Seepage control measures appeared to be functioning as designed. The structural integrity and support of the spillway is questionable and requires attention to insure flows do not adversely affect embankment safety. There is no riprap or erosion protection on the embankment slopes. Rill erosion is evident but is controlled by seasonal maintenance. Review of the stability analysis on file with the owner indicates the analysis is adequate and that Zonolite Tailings Dam conforms with inspection guideline stability criteria.

RECOMMENDATIONS

Develop and immediately place in action a downstream warning plan for use in the event of possible dam overtopping or structural failure. Periodically test the decant line for possible leaks within the embankment and perform necessary maintenance and repairs. Conduct more detailed hydrologic and hydraulic routing studies to better determine downstream hazard potential and to establish the minimum safe flood storage volume and spillway capacity. Studies should take into account the continually decreasing water storage volume as tailings are deposited, the nature of the tailings and their effect of sudden release on the downstream environment and an evaluation of the structural adequacy of the spillway under a full range of possible flow conditions. Modify the operation and/or project as studies indicate. Continue to monitor and evaluate seepage and conduct periodic inspections of the project on at least an annual basis by engineers experienced in dam design and construction.

Prior to performing engineering studies or remedial construction, coordinate with applicable Federal and State agencies to insure compliance with all pertinent laws and regulations.

MONTANA
RODGER C.
FOSTER
3654 E
REGISTERED
PROFESSIONAL ENGINEER

Rodger C. Foster
Professional Engineer

PERTINENT DATA
ZONOLITE TAILINGS DAM

1. GENERAL

Federal ID No.	MT-1470
Owner	W.R. Grace & Company
Operator	W.R. Grace & Company
Date Constructed	Original 1971 Expanded 1973, 1975, 1977, 1980
Location	Section 22, T31N, R30W Longitude 115°24'40" Latitude 48° 26'32"
County, State	Lincoln County, Montana
Watershed	Rainy Creek
Size	Large
Downstream Hazard Potential	Category 1, High
USGS Quadrangle	Vermiculite Mountain, Montana

2. RESERVOIR (Phase 5)

Surface Area at Spillway Crest	68.5 acres
Drainage Area	9.7 square-miles
*Storage at Spillway Crest (elevation 2920 feet NGVD)	2120 acre-feet
*Storage at Dam Crest (elevation 2925 feet NGVD)	2450 acre-feet
*Storage at Decant Line Inlet (elevation 2890.1 feet NGVD)	550 acre-feet
Surcharge Storage	330 acre-feet
Reservoir Elevation (25 July 80)	2880.0 feet, NGVD
during construction of Phase 5 (13 August 81)	2893.8 feet, NGVD

*Water storage only
(Assumed tailings elevation 25 July 80, 2873.0 ft.)

3. SPILLWAY (Phase 5)

Type	Uncontrolled chute
Shape	Half section, 8-foot diameter round corrugated metal pipe.
Crest Elevation	2920 feet, NGVD
Capacity with Reservoir at Dam Crest	200 cfs

4. OUTLET WORKS/DECANT TOWER (Phase 5)

Decant Tower	6-foot diameter steel pipe, set vertically with a 1.7-foot wide entrance extending the full height of pipe. Floor elevation at 2890.1 feet.
Decant Line	1600 feet of 16 inch diameter welded steel pipe at 1% slope.
Gate	Uncontrolled overflow weir to wet well inlet structure.
Capacity with Reservoir at Dam Crest:	18 cfs

5. DAM (Phase 5)

Type	Compacted Tailings
Length	1154 feet at elevation 2925 feet NGVD
Crest Width	40 feet at elevation 2925 feet NGVD
Crest Elevation	2925 feet NGVD
Hydraulic Height crest to toe	135 feet
Upstream Slope	1V on 2H
Downstream Slope	1V on 2H w/Construction Terraces

Chapter 1

BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority and Scope

This report summarizes the Phase I inspection and evaluation of the Zonolite Tailings Dam, owned by W.R. Grace and Company, Construction Products Division.

The National Dam Inspection Act, Public Law 92-367 dated 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-Federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many Federal and State agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive consensus of the engineering community.

Where necessary, the guidelines recommend a two-phased study procedure for investigation and evaluation of existing dam conditions, so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam.

(2) based upon an evaluation of the available data and a visual inspection.

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations, and analyses are necessary or warranted.

(4) not intended to include extensive explorations and analyses or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses, and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally-owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases, the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise. The two phases of investigation outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective work.

The responsibility for implementation of these Phase I recommendations rests with the dam owner and the State of Montana. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this Act shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

1.1.2 Purpose

The purpose of the inspection and evaluation is to identify conditions that threaten public safety, so that they may be corrected in a timely manner by non-Federal interests.

1.1.3 Inspection

The findings and recommendations in this report are based on brief visual inspections of the project and a detailed review of available construction plans, analyses and reports. Inspection procedures and criteria are those established by the Recommended Guidelines for Safety Inspection of Dams (Reference 1).

Personnel present during the 25 July 1980 inspection included:

Larry Tegg,	State of Montana, Department of Natural Resources and Conservation
Rodger Foster,	Team Leader, Morrison-Maierle, Inc. Water Resource Engineer
Mike Kaczmarek,	Engineering Geologist, Morrison-Maierle, Inc.
Robert Foss,	Chief Engineer for W. R. Grace and Company, Zonolite Operations at Libby.

Those present for the 13 August 1981 inspection were:

Art Taylor,	State of Montana, Department of Natural Resources and Conservation
Harold Eagle,	Chief Engineer, Morrison-Maierle, Inc.
Mike Kaczmarek,	Engineering Geologist, Morrison-Maierle, Inc.
Phil Porrini,	Water Resources Engineer, Morrison-Maierle, Inc.
Michael Ray,	Chief Engineer for W.R. Grace and Company, Zonolite Operations

Additional Morrison-Maierle personnel who contributed to the evaluation are:

Bill Keith, Structural Engineer

Ken Salo, Hydrologist/Hydraulics Engineer.

Subsequent discussions and coordination were conducted with Mr. Lyle Lewis of Harding-Lawson Associates and Messers William McCaig and Michael Ray of W.R. Grace Company, concerning the completed Phase 5 addition to the dam.

This report was reviewed by W.R. Grace and Company-Construction Products Division, the Montana Department of Natural Resources and Conservation, the Mine Safety and Health Administration and Mr. Lyle Lewis of Harding-Lawson Associates. Mr. Lewis submitted verbal comments only. The written comments received are included in the appendix.

1.2 DESCRIPTION OF PROJECT

1.2.1 General

Zonolite Tailings Dam and Reservoir are located at the site of the W.R. Grace and Company vermiculite mine on Rainy Creek about 6.5 miles northeast of Libby, Montana (Plates 1 and 2) (Photos 1 and 2). Rainy Creek empties into the Kootenai River 5.5 miles upstream of Libby. The project's Federal identification number is MT-1470. The 135-foot high tailings dam creates a reservoir that normally stores only enough water to allow mine tailings to settle out (about 7 feet deep). Normal flow from the Rainy Creek drainage basin is diverted around the reservoir through a 48-inch diameter corrugated metal pipe (CMP) (Photo 5). With the tailings at assumed elevation 2,873.0 feet and the dam crest at elevation 2,925 feet, the dam is capable of impounding about 2,450 acre-feet of water to the dam crest. As tailings accumulate, storage volume decreases. In time, the dam is raised to provide needed storage. The dam is planned to eventually be 200-feet-high. Based on visual reconnaissance and engineering judgement, the screening plant and product storage area at the mouth of Rainy Creek and State Highway 37 could be affected by a sudden breach of the tailings dam. In accordance with the Recommended Guidelines, the project is large in size and the downstream hazard potential is high (Category 1).

Outlets from the reservoir consist of an uncontrolled chute spillway in the left abutment and a decant structure located in the upper reservoir area (Photos 3 and 6). The spillway consists of a trapezoidal approach channel which transitions to a half-section of an 8-foot diameter corrugated metal pipe. The decant structure is a weir-controlled inlet structure to a 16-inch diameter welded steel pipe which extends along the west shoreline in the reservoir, through the dam embankment and discharges into the natural drainage downstream of the dam. The decant tower is designed to skim the cleaner surface water from the tailings pond.

1.2.2 Regional Geology and Seismicity

The Zonolite Tailings Dam is located in the northwest corner of Montana in the Rocky Mountain physiographic province (Reference 2). The area is characterized by high, rugged north-northwest-trending mountain ranges separated by narrow linear valleys that parallel the ranges. The mountain ranges are composed of late Precambrian Belt Series strata consisting of fine-grained clastic and carbonate rocks.

The Belt Series rocks range from 17,000 to 40,000 feet thick (Reference 3) and have undergone regional low-grade metamorphism. The Belt Series strata near Libby and the Zonolite Tailings Dam are deformed into broad, open, north-northwest-trending folds. High angle normal faults of regional proportions parallel the trend of the folds. The narrow linear valleys of the region are fault-bounded structural troughs. Surficial deposits are present on the floor of the Kootenai River valley and its major tributaries and consist primarily of alluvial sand and gravel, glacial lake silts, and alpine glacial deposits.

The Zonolite Tailings Dam is located on an elongate intrusive rock body that intrudes strata of the Precambrian Wallace Formation within the trough of a north-west-trending synclinal fold. The intrusive body is named the Rainy Creek stock and is a complex, composite intrusive which encompasses about 7.5 square miles of outcrop area underlying Vermiculite Mountain and most of the Rainy Creek valley west of Vermiculite Mountain. Vermiculite is mined from this intrusive.

In accordance with the Guidelines' Seismic Zone Map (Reference 1), the Zonolite Tailings Dam is in Seismic Zone 2. The seismic probability of Zone 2 has a potential for moderate earthquake damage and is based on known distribution of damaging earthquakes. Stability analysis performed for the planned 200 foot high embankment indicate that the computed safety factors exceed minimum recommended allowable safety factors for static conditions and also for seismic loadings of 0.1 and 0.2 times gravitational acceleration (Reference 6). Recommended Guidelines indicate that no hazard to embankment dams from earthquakes generally exists in Seismic Zone 2 provided static conditions are satisfactory and conventional safety margins exist.

1.2.3 Site Geology

The geology along the foundation of the Zonolite Tailings Dam was explored by means of 8 test borings 29 to 55 feet deep, 14 test pits 7 to 20 feet deep, 3 seismic velocity survey lines, and 27 soils resistivity soundings all conducted under the supervision of Harding-Lawson Associates in December, 1970 and May, 1971. The seismic velocity lines were used to determine depths to bedrock below the unconsolidated surficial deposits in the foundation area. The resistivity soundings were used to confirm the continuity of unconsolidated deposits between test boring locations. Field techniques and general methodology used are described in Appendix B of Lewis and Lawson (Reference 6).

A summary description of the preconstruction site geology is provided in Appendix A of Lewis and Lawson (Reference 6). Field observations conducted at the time of the dam safety inspection in the area of the abutments and foundation were in agreement with the 1971 geologic report. The July 25, 1980, field inspection revealed bedrock Underlying the dam site in the left abutment consists of the magnetite pyroxenite described by Boettcher (1963). The bedrock in the foundation area and in the right abutment area was covered by alluvium and glacial deposits and was not observed during the inspection. The bedrock exposed on the right abutment above the elevation of the existing dam (elevation 2890 feet) appeared to be the least altered, finer-grained pyroxenite of Boettcher (1953).

The magnetite pyroxenite observed in construction cuts on the downstream side of the left abutment on July 25, 1980, was a highly weathered, friable rock. As described in Appendix A of Lewis and Lawson (Reference 6), the "upper few feet of pyroxenite bedrock has physical characteristics more like those of a dense sand than rock." Discontinuous shear planes 2 to 3 inches thick oriented parallel to the valley wall were observed in the left abutment rocks on about 2 to 3 foot spacing. Lewis and Lawson (Reference 5) interpret the thin shear zones, which contain silt and clay sized rock gouge, to be the result of either glacial ice loading and/or gravity induced rock creep. The left abutment bedrock is discontinuously covered with as much as 10 feet of weathered rock debris. Lewis and Lawson (Reference 6) describe about 4 feet of highly permeable clean sand and gravel present as an outwash terrace remnant at about elevation 2830 feet.

Test pits and borings show the unconsolidated surficial deposits in the right abutment area consist of glacial outwash and till as much as 40 feet thick up to about elevation 2,890 feet (Reference 6). Cuts on the right abutment near elevation 2,870 feet on July 25, 1980, exposed fluvially bedded gravelly coarse sand (outwash). Subsurface investigations (Reference 6) of the unconsolidated foundation materials show as much as six feet of soft silt with lenses of fine sand and sparse gravel stringers over coarse gravel outwash with lenses of silt and fine sand and zones of quartzite boulders 4 to 5 feet in size. The depth to bedrock below the land surface on the alluvial silts and the gravel outwash ranges from 26 to 45 feet in the test boring logs (Reference 6).

1.2.4 Design and Construction History

The Zonolite Tailings Dam was designed in 1971 for W. R. Grace & Company by Bovay Engineers, Inc. of Spokane, Washington and Harding-Lawson Associates of Novato, California. The dam was designed as a tailings impoundment dam to retain fine tails produced in the vermiculite milling process. The staged construction plan called for the construction of a starter dam with provisions for raising the dam in stages as the storage capacity was depleted by tailings. The starter dam was constructed immediately downstream of an older existing dam. This older dam is identified in Reference 6 by crest elevation (2830 feet) and centerline location.

The 50-foot high starter dam was completed in November 1971 to an elevation of 2850 feet. Since that time three additional phases have been completed. Under the phase 1 expansion, completed in June 1973, the dam was raised to elevation 2875 feet. Phase 2 was completed in 1975 and involved raising the dam approximately 5 feet to elevation 2880 feet.

Phase 3 work was begun in late September 1976 and completed in August of 1977. Phase 3 construction brought the dam to elevation 2890 feet, which was the crest elevation at the time of the July 25, 1980 inspection. The Phase 4 addition, was delayed, and later incorporated in the Phase 5 construction. The Phase 5 addition was under construction at the time of the July 25, 1980 inspection, and was completed in October 1980, which raised the dam to elevation 2925 feet. While the phased construction of the dam has added a total of 75 feet to the 50-foot high starter dam's crest, it has also included approximately 10 feet of elevation difference at the downstream toe, making the total hydraulic height of the Phase 5 dam 135 feet.

A log of the planning and construction of the dam including construction drawings is on file with the chief engineer at the W. R. Grace mill site. Drawings showing the "in-place" construction phases are presented on Plates 4 & 5. A stability analysis for the planned 200 foot high embankment is on file with the owner. During construction of the various phases, Bovay Engineering (for the starter dam) and Harding-Lawson provided engineering services that included observation, consultation and material testing. Records indicate embankment construction met or exceeded design criteria.

Information in the Foundation Investigation and Engineering Analysis, (Reference 6) is cited throughout this Phase I - Dam Safety Report as it contains detailed information regarding the design of the starter dam and discussion of the planned 200 foot high embankment.

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway (Photos 6 to 13) is located in the east abutment and has been built to the same design criteria as described for the Phase 3 spillway in a June 8, 1976, letter to Robert Foss from Lyle Lewis of Harding - Lawson Associates and in a June 29, 1977, letter to Mr. Purnel Whitehead of W.R. Grace & Co. from Lyle Lewis concerning spillway design data (Reference 7). The spillway design criteria is presented on Plate 7. To our knowledge, the spillway has never been operated.

The spillway consists of an unlined trapezoidal shaped approach channel with a concrete transitional inlet structure which leads to half-sections of 8-foot diameter corrugated metal pipe (CMP) that comprise the spillway chute. Two pipes carrying mine tailings cross the unlined trapezoidial channel only slightly above the invert elevation (Photo 8). These pipes would trap floating debris during periods of spillway use and reduce its capacity. Bank sloughing and deposition of material in this unlined approach channel also represents serious capacity reductions and should be prevented. The entrance elevation of spillway is approximately 5 feet below the dam crest. The trapezoidal channel which extends through the east abutment abruptly transitions to the CMP chute (Photos 6 and 7). The CMP chute meanders along the haul road near the left abutment at an eight percent slope for approximately 500 feet and discharges into an unlined roadside ditch. Discharges enter the natural drainage downstream of the embankment toe. No stilling basin or energy dissipater is provided for at the terminus of the spillway chute (Photos 12 and 13). The CMP is anchored to concrete collars at three points along its length (Photo 10) and is also anchored to the concrete trapezoidal section near the crest. Pipe sections are over-lapped and bolted to each other but are not sealed to prevent seepage which could produce piping of foundation materials and cause the chute to fail (Photo 9). This assemblage of the spillway raises doubts that the spillway chute will safely handle any high flows. Because of the spillway location, the ability of the half-round pipe to withstand hydraulic forces under all flow conditions, is an important consideration to the safety of the dam.

There was no floating debris noted in the pond, however, the drainage basin and much of the shoreline is heavily timbered and debris could be easily carried into the pond during high flows. There are no apparent provisions to protect the spillway from floating debris.

A spillway rating (Plate 8) was developed using the HEC-2, Water Surface Profiles (Reference 8). Backwater computations determined that critical depth occurs at the spillway crest entrance (elevation 2,920 feet NGVD). A Mannings "n" of 0.022 was used. The maximum discharge capacity of the spillway with the reservoir at the assumed dam crest elevation 2,925 feet NGVD, was estimated to be 200 cfs or about 17 acre-feet per hour. The two slurry pipes which cross the unlined trapezodial channel were not considered in the capacity analysis.

2.1.2 Outlet/Decant Tower

Flow from the pond under normal operations is controlled by the decant tower. The Phase 5 decant tower is located approximately 1000 feet upstream of the dam (Plate 3) and consists of a 72-inch diameter corrugated metal riser pipe with a 1.7-foot wide rectangular weir entrance with a minimum elevation at 2890.1 feet, NGVD (Photo 3). The maximum weir elevation is the top of the decant tower at approximately 2923.0 feet, NGVD. The purpose of the decant tower is to maintain the level of the settling pond so that solids from the mill tailings are settled out and only the clean surface water is discharged through the decant line. The pond elevation is controlled by stoplogs placed in the decant tower entrance. Stoplogs acting as a weir crest will be installed permanently in stages as tailings accumulate in the pond to maintain several feet of water above the tailings. A log boom (Photo 3) in front of the stoplogs prevents floating debris from clogging the decant tower.

The Phase 5 decant line extends from the tower along the floor of the pond and through the dam embankment discharging approximately 500 feet downstream of the dam. This line consists of 16-inch diameter welded steel pipe. The pipe drains directly from the floor of the decant tower. Mr. Ray stated the portion of the decant line located within the embankment is encased in 1 foot of concrete with two cutoff collars for seepage control. The normal discharge capacity of the decant line was estimated to be 5 cfs and could increase to 18 cfs under unobstructed pressure flow conditions.

2.1.3 Rainy Creek Diversion

Under normal operations Rainy Creek is diverted around the dam. Fleetwood Creek, a tributary to the tailings pond, is not diverted and does enter the reservoir. A 48-inch diameter CMP diversion pipe with an approximate capacity of 100 cfs has been constructed to convey flow from Rainy Creek upstream of the reservoir above the 3,000 foot elevation and divert the flow downstream of the dam (Photo 5). A diversion structure intercepts normal runoff from 60 percent of the drainage basin and discharges it into the diversion pipeline. If inflows were to exceed the diversion pipeline capacity and/or the crest elevation of the flashboard assembly in front of the overflow culvert, then the remaining flow discharges directly into the tailings pond. At the August 13, 1981 inspection, the elevation at the top of the flashboards for the overflow culvert was approximately six inches higher than the water surface in front of the diversion structure, and approximately three feet lower than the structure's crest elevation (Photo 4).

2.1.4. Freeboard

This study indicates the dam overtops during the recommended spillway design flood (SDF) which is the probable maximum flood (see paragraph 2.2.4). Therefore, it has no freeboard. At the time of the July 25, 1980 inspection, the vertical distance from the water level (approximate elevation 2,880 feet) to the dam crest (elevation 2,890 feet) was approximately 10 feet. After the Phase 5 construction, and during the August 13, 1981 inspection, the normal water level was 31 feet below the dam crest. However, the vertical distance between normal water level and the top of the dam will become less as tailings accumulate in the reservoir. Historically the dam has been raised to provide additional storage when the normal reservoir operating level came to within 10 to 12 feet of the dam crest. The spillway crest on the Phase 5 addition is five feet below the crest of the dam. The dam is located on the southwest end of the pond and the prevailing winds would be directed away from the dam. The effective fetch for wind-generated waves resulting from a north wind is about 2,000 feet and wave run-up on the embankment is estimated to be less than 3 feet. Although the dam will be overtopped by the PMF, the vertical distance between the normal pool elevation and the dam crest is adequate to prevent overtopping of the embankment by wind-generated waves.

2.2 HYDROLOGY, CLIMATOLOGY AND PHYSIOGRAPHY

2.2.1 General

The climate of the area is continental in nature characterized by warm summers and cold winters. Summer temperatures rarely exceed 95°F, and winter temperatures can reach 25 to 30 degrees below 0°F. Winters have few extended extreme cold spells due to periods of warm "chinook" winds.

The Rainy Creek drainage is located between two climatological stations. The Libby 1 N.E. Ranger station site is located 5.5 miles west southwest at an elevation of 2,080 feet NGVD. The Libby station has 74 years of record for temperature and 84 years of record for precipitation. Mean annual precipitation at Libby is 19.4 inches with 37 percent of it occuring in the months November through January and 18 percent falling in the months of May and June. The month having the highest average precipitation is January with 2.42 inches. Temperatures at Libby range from an average of 22.4° Fahrenheit (F) in January to an average of 67°F in July. May and June temperatures average 54°F and 60.3°F, respectively.

A second climatological station is located approximately 5.1 miles east southeast of the basin at Libby Dam. The Libby Dam station is at elevation 2,200 feet NGVD and has 12 years of record for temperature and precipitation.

Average annual precipitation in the Rainy Creek drainage is estimated to be 30 inches per year (Reference 9) and the temperatures would be expected to average 3 to 5 degrees cooler than at Libby.

The Rainy Creek drainage basin above the tailings dam is 9.7 square miles in area and is generally "L" shaped. It is located on a southern exposure of the Purcell Mountains and is primarily forest covered except for the mine area. The basin rises from an elevation of 2,800 feet at the dam to 6,040 feet at Blue Mountain in a stream length of approximately 4.6 miles. There is no gaging station in the basin and the nearest gage downstream is on the Kootenai River at Libby (U.S. Geological Survey Station No. 12303000).

A hydrology report on the Rainy Creek drainage basin was prepared for W.R. Grace and Company in February 1971 by Bovay Engineers, Inc. and indicates the maximum design flow in Rainy Creek at Fleetwood Creek at 200 cfs. The maximum design flow is based on comparisons of maximum discharge of record, per square mile of drainage area with six nearby and hydrologically similar gaging stations.

2.2.2 Reservoir Storage

In estimating the storage volume of the tailings pond it was assumed that mill tailings deposits had filled the pond to an elevation of 2,873 feet which means the water was approximately 7 feet deep at the time of the inspection. Using the conic method to develop total storage volume for the completed Phase 5 it is estimated that the reservoir would have a surface area of 68.5 acres and a volume of 2,120 acre-feet with the pool at the spillway crest elevation 2,920 feet. Approximately 330 acre-feet of surcharge storage is available in the reservoir between the spillway crest and the dam crest.

Based on estimates of anticipated plant production, tailings fill the reservoir storage at a rate of 135 acre-feet per year. Continued reduction in reservoir floodwater storage by deposition of tailings will seriously affect flood handling potential. Historically the dam has been raised to provide additional storage when the normal reservoir operating level came to within 10 to 12 feet of the dam crest.

2.2.3 Estimated Probable Maximum Flood

The probable maximum flood (PMF) is the flood expected from the most severe combination of critical meterologic and hydrologic conditions that are reasonably possible in the region. An estimate of the PMF was made during this dam safety analysis and was routed through the reservoir.

The probable maximum precipitation (PMP) was developed using a procedure contained in the U.S. Weather Bureau's Hydrometerological Report No. 43 (Reference 10) as updated by U.S. Weather Bureau memorandum dated 9/20/67 (Reference 11). The storm which produces the PMF would be a 6-hour thunderstorm during the period July to August. The July-August thunderstorm PMP produces 6.0 inches of rain in one hour and 8.0 inches of rain in six hours. A minimum loss rate of 0.15 inches per hour was assumed to represent the hydrologic class B soils in the basin and minimum infiltration conditions due to saturated ground. Baseflow was considered to be 110 cfs from Rainy Creek and Fleetwood Creek.

A triangular unit hydrograph for a 10-minute rainfall duration was developed for the 9.7 square mile drainage area using procedures contained in Design of Small Dams (Reference 12). The Soil Conservation Service method of developing a curvilinear fit of the triangular unit hydrograph was used. The hourly increments of the PMP were arranged in a critical time sequence as illustrated in HMR No. 43 page 181 (Reference 10). The 10-minute increments from the greatest two hours were rearranged in the reverse order of the unit hydrograph to produce the greatest possible peak. The PMP was applied to the unit hydrograph by means of the computer program HEC-1 (Reference 13). This estimate of the PMP produced a flood with a peak flow of 43,400 cfs and a volume of 3,770 acre-feet.

2.2.4 Flood Routing

Routing of the probable maximum flood through Zonolite Tailings Pond was performed using the computer program HEC-1 Flood Hydrograph Package (Reference 13). The reservoir routing was started at the minimum elevation of the decant tower (2890.1 feet NGVD) which is the current operating level. This elevation, however, will change as stoplogs are permanently installed to keep water above the tailings. A 100-year 24-hour antecedent storm of 3.4 inches obtained from the National Oceanic and Atmospheric Administration (NOAA) together with Soil Conservation Service (SCS) runoff curve number (CN = 55, forested, class B soil) produced 0.35 inches of runoff. It was assumed that the diversion structure on Rainy Creek had failed allowing the entire runoff from Rainy Creek to join Fleetwood Creek and enter the reservoir. The total volume entering the reservoir would be approximately 180 acre-feet and would raise the reservoir elevation four feet to 2894.0 feet. The decant line discharging at 10 acre-feet per day would leave the reservoir at nearly the same elevation (2894.0 feet NGVD). This elevation was used to start the PMF routing.

Information from the owner regarding reservoir levels indicates that the drainage basin which is not controlled by the Rainy Creek diversion has never seriously contributed to raising the pond elevation. Only the installation of flashboards in the decant tower causes changes in pond elevation. There is no record of either Phase 3 or Phase 5 spillway use.

Routings indicate that the dam overtops during the PMF when approximately 55 percent of the total flood volume enters the reservoir. Routings were made of lesser hypothetical floods than the PMF to determine the magnitude of floods the dam can contain. The hypothetical hydrographs are obtained by applying percentages to the PMF hydrograph ordinates. A flood with a hydrograph having ordinates corresponding to 45 percent of the PMF ordinates is just controlled by the project. Larger floods would overtop the dam. Depletion of water storage volume by adding mill tailings would further reduce project flood handling capabilities.

2.3 GEOTECHNICAL EVALUATION

The following geotechnical evaluation is based on field inspections of the project, examination of the referenced reports, plans, and specifications, and discussion of the construction history with Bob Foss and Mike Ray, Chief Engineers for Zonolite Operations of W.R. Grace and Company, and Lyle Lewis, consulting design engineer, Harding-Lawson Associates.

2.3.1 Dam

The Zonolite Tailings Dam, is a homogenous fill resulting from several phases of incremental construction. The different phases of incremental construction of the embankment are shown on Plate 5. Phases 4 and 5 were constructed simultaneously. The elevation at the base of the dam is about 2,790 feet. The initial starter dam crest elevation was 2,850 feet and successive increments were added at elevations 2,875, 2,880, and 2,890. The phase of construction ongoing at the time of the July 25, 1980 inspection increased the dam height 35 feet to elevation 2,925 feet.

Each of the increments were added to the dam embankment using a downstream method of construction with slopes of 1V on 2H for upstream and downstream faces, and intermediate benches on the downstream face as shown on Plate 5. Table 1 shows a summary of dam height, crest length, and crest width for each successive increment of dam height. The final downstream slope configuration for the structure at elevation 2,925 feet NGVD including erosion benches is shown on Plate 5 and Photos 6, 14, 15 and 16.

Table 1: Summary of general dimensions for incremental structures.[1/]

ELEVATION (Feet)	DAM HEIGHT (Feet)	CREST WIDTH (Feet)	CREST LENGTH (Feet)
2850 Starter Dam	50	40	840
2875 Phase 1	75	40	975
2880 Phase 2	80	35	995
2890 Phase 3	95	22	1002
2899 Phase 4*	105	80	1055
2925 Phase 5	135	40	1154

[1/]Crest lengths and widths scaled from W.R. Grace & Co. Construction Products Division Drawing No. 40-1009 as revised for each construction phase (Plate 4).

*incorporated in Phase 5 construction.

The materials used to construct the embankment are old mill tailings obtained from a stock pile on the east side of the tailings retention pond area, soils stripped from the abutment areas, and gravel from the location of the old mill pond downstream from the existing structure. Memos and letters in the Harding-Lawson Associates job file indicate that embankment fill was placed in three- to six-inch lifts and compacted to 95 percent relative compaction as determined by laboratory test procedures and periodic field density tests. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-70(C) laboratory test procedures. Records of extensive field density tests during construction phases (thru Phase 3) show relative compaction of 95 percent or more (Reference 7). Discussion with the design consultant indicates that field density tests during Phase 5 construction also conformed with design criteria.

The main bulk of the embankment consists of mill tailings. Preconstruction tests show the mill tailings to consist of greenish gray gravelly sand with a dry density of about 138 pounds per cubic foot (S.G.=3.1), a permeability of 1×10^{-3} feet per day at 95 percent relative compaction, an internal angle of friction (ϕ) of 20°, and cohesion (c) of 4000 psf, (Reference 6 and Reference 14). Strength parameters of ϕ and c are based on consolidated-undrained triaxial tests with pore pressure measurements. Maximum dry density data for mill tailings provided for the field density tests range from 144 to 149 pounds per cubic foot. Tests show surficial materials on the abutments to be gray silty sandy gravel and sandy gravel that mix to a dry density of 128 pounds per cubic foot and exhibit a permeability of 2 x 10^{-5} feet per day at 95 percent relative compaction (Reference 6).

Inspection of the embankment did not reveal any sign of cracking, differential settlement, misalignment, or slope failure. A review of the extensive construction inspection records (Reference 7) and discussions with the W.R. Grace Engineer indicate that there have been no stability or consolidation problems with the embankment.

There is no protective shell or riprap on either face of the embankment. However, the coarse-grained mill tailings slurry discharged at the upstream face of the embankment, prohibits serious wave erosion of the embankment face (Photo 17). The downstream face of the dam has been under intermittent construction since 1971. There was evidence on both inspections that rain storms had caused accelerated erosion on the downstream dam slope (Photo 15). Bob Foss stated that rill erosion between benches on the downstream face demands seasonal maintenance by face dressing. A new surface drainage system consisting of half-sections of 42-inch diameter CMP has been constructed in the left abutment (Photo 14). This system drains the bench cuts on the downstream face of the Phase 5 addition. Erosion on the dam face does not pose a hazard to the dam so long as timely maintenance is performed and erosion control measures such as bench cuts or erosion terraces are used. During the PMF, rainfall intensity of 8 inches in 6 hours may cause extensive erosion on the dam's face.

2.3.2 Foundation Conditions, Seepage, and Drainage

The foundation of the Zonolite Tailings Dam, as determined by test borings, test pits, resistivity soundings, and seismic velocity measurements (Reference 6) consists of 20 to 40 feet of sand and gravel outwash with interbedded lenses of fine sand and silt all resting on weathered pyroxenite bedrock. Unsuitable materials were stripped from the foundation. Embankment fill was placed on dense gravelly soils containing abundant cobbles and boulders following foundation stripping. Foundation stripping and preparation was accomplished in phases corresponding to the phases of embankment construction.

Abutment preparation prior to placement of compacted fill consisted of constructing bench cuts 4 to 6 feet wide into the abutments. Bench cuts on the left abutment are in weathered, friable magnetite pyroxenite as observed during the July 25, 1980 inspection and documented in construction inspection memos. Bench cuts on the right abutment required 10 to 15 feet of excavation to remove loose slope debris so that the bench cuts key into the dense glacial soils. Construction inspection memos and as-built drawings for the construction increment bringing the dam crest to elevation 2,890 feet NGVD, describe right abutment bench cuts keyed into glacial soils below elevation 2,860 feet and bedrock above that elevation. At the time of the July 25, 1980 safety inspection, the materials exposed by bench cuts on the right abutment above elevation 2,890 feet consisted of firm glacial soils.

Construction inspection memos and letters (Reference 7) beginning October 24, 1972 describe the presence of an adit in the right abutment in the Rainy Creek ditch cut below elevation 2,875 feet. Discussions with Lyle Lewis of Harding-Lawson Associates revealed the adit was backfilled and drain pipes were installed to control seepage. The absence of seepage in the right abutment contact area and the width of the embankment fill in the reported location of the adit suggest the adit does not pose a threat to the abutment integrity.

Drainage installations employed at the Zonolite Tailings Dam consist of a chimney drain in the original starter dam and foundation drains consisting of perforated concrete pipe bedded in pervious aggregate. The location of foundation drains and the location of the chimney drain, which was not extended beyond the starter dam, are shown on Plates 4 and 5. The foundation drainage system presently consists of two cross drains and essentially seven lateral drains (Photo 12 to 25).

The starter dam was constructed immediately downstream of an older existing dam, which was reported to have a crest elevation of 2830 feet. A foundation cross drain was placed roughly from abutment to abutment and located between the upstream toe of the starter dam and the older existing dam. The cross drain is indicated in the foundation drain pattern on Plate 4 and is composed of a 10 foot wide by one foot thick bed of select drain material. This upstream toe drain is covered by compacted mill tailings used to fill the space between the starter dam and the old existing dam. The so called toe drain discharges into two of the lateral drains under the starter dam embankment.

Construction memos (Reference 7) indicate that toe drains 18 to 24 inches wide and 36 to 48 inches deep were placed at the toe of the starter dam and the first addition to the dam as trench drains to intercept seepage from the natural foundation gravels observed during construction. The toe drains were limited to the abutment portion of the starter dam downstream toe. Foundation stripping for the first addition to the starter dam revealed a spring in the foundation. The spring was excavated and filled with pervious gravel and is drained by the 14 inch steel pipe shown on Plate 4 and Photos 19, 21 and 22.

The foundation drainage system shown on Plate 4 was extended and in some areas expanded as part of the Phase 5 addition. Five perforated concrete drain pipes bedded in gravel were inspected (July 80 and Aug. 81) and each revealed clear water either seeping from the drain pipe or bedding material (Photos 19 to 25). A wetted front along the downstream toe was observed during the August 13, 1981 inspection and extended between all drains at approximate elevation equal to the flow line of the drains. The two 10-inch drain pipes closest to the spillway shown on Plate 4 do not extend to the embankment toe. These two drain pipes are buried but are scheduled to be uncovered by the owner. The 14 inch steel pipe draining the spring in the foundation was discharging flow estimated to be 100 to 200 gpm (Photo 22).

Piezometer monitoring data are available for each phase of dam construction. Five piezometers in the completed Phase 5 addition are shown on

Photo 17 and are located on Plate 5. These piezometers are merely vertical extensions of those that were in place for Phase 3. Only seven readings of the Phase 3 piezometers were recorded in a 29 month period between December 1977 and May 1980. Readings of the water level in the open tube piezometers should be correlated to tailings pond levels but records do not indicate such data is being kept. Although limited information has been obtained since the completion of Phase 5, there continues to be no evidence of significant water levels above the foundation.

Additional piezometer data was collected during the August 13, 1981 follow-up inspection. Measurements included elevations at each piezometer casing top, depths to water from the piezometer casing tops, and total piezometer depths as measured from the casing tops. Table 2 shows elevation and depth measurements conducted on August 13, 1981 and compares measured bottom elevations for each of the piezometers to bottom elevations shown on the as-built drawings (Plate 5).

TABLE 2: Measured piezometer elevations and depths compared to as-built data.

Piezometer No. (Plate 5)	Measured Top Casing Elev.1/ (Feet)	Measured Piezometer Depth (Feet)	Measured Bottom Elev. (Feet)	As-Built Bottom Elev. (Feet)	Difference From As-Built (Feet)
#1	2,920	104	2,816	2,807	9
#2	2,920	121	2,799	2,790	9
#3	2,920	60	2,860	2,795	65
#4	2,921	106	2,815	2,806	9
#5	2,921	104	2,817	2,811	6

1/ Measured elevations based on assumed crest elevation 2,925 at centerline of dam crest at midpoint between abutments and August 13, 1981 survey. Elevations and depths rounded to nearest foot.

Table 2 shows that measured elevations for the bottoms of the piezometers as of August 13, 1981 are higher than those shown on the as-built (Plate 5). Piezometer #3 in particular is 65 feet shallower than originally constructed. W. R. Grace and Company Chief Engineer for Zonolite Operation, Michael Ray, states that there are indications that vandals have been dropping rocks and loose earth down the piezometers. Measurements in piezometer #3 on August 13, 1981 showed considerable dirt sticking to condensation on the side of the casing beginning at about 34 feet from the top of casing and persisting to the bottom at above 60 feet below the casing top. The foregoing observations suggest that the differences between measured and as-built piezometer depths and bottom hole elevations (Table 2) may possibly be attributed to rocks and earth dropped into the piezometers.

Table 3 shows the depths to water and water surface elevations measured in the piezometers on August 13, 1981. Water surface elevations are compared to the as-built foundation elevation shown on Plate 5. Data on Table 3 show a phreatic surface in the lowermost 22 feet and 15 feet of the dam embankment at piezometers #1 and #2, respectively. Both piezometers #1 and #2 bottom out in the embankment fill without penetrating foundation material. Piezometer #3 is too plugged to yield useful data. Piezometers #4 and #5 both penetrate foundation materials and indicate that the phreatic surface in the foundation at these locations is at a lower elevation than the base of the embankment. The data shown on Table 3 indicate that the foundation drains and the pervious alluvial foundation materials are effectively controlling embankment and foundation seepage within conservatively safe levels.

TABLE 3: Water surface elevations measured in piezometers on August 13, 1981.

Piezometer No.	Measured Top Casing Elev.1/ (Feet)	Measured Depth to Water2/ (Feet)	Water Surface Elev.2/ (Feet)	As-Built Foundation Elev. (Feet)	Pond Elev.3/ August 13, 1981 (Feet)
#1	2,920	102.03	2,818	2,796	2,894
#2	2,920	114.76	2,805	2,790	2,894
#3	2,920	DRY	DRY	2,796	2,894
#4	2,921	DRY	DRY	2,812	2,894
#5	2,921	103.89	2,816	2,841	2,894

1/ Measured elevations based on assumed crest elev. 2,925 at centerline of dam crest at midpoint between abutments and August 13, 1981 survey. Elevations and depths rounded to nearest foot.
2/ Water levels measured by steel tape and chalk method to ±0.01 feet.
3/ Water surface elevation rounded to nearest foot.

2.3.3 Stability

A design stability analysis for the proposed 200-foot high tailings embankment is on file with the owner. Since all lower dams are similar to the planned 200-foot high embankment, consist of homogenous coarse tailings, and have 1V on 2H upstream and downstream slopes and a 20-foot minimum crest width, no stability analyses were performed for individual construction phases. The design stability analysis used an assumed phreatic surface and shear strength data developed from testing of embankment mill-tailing materials.

Results of the field testing through Phase 3 construction were reviewed and are in conformance with the design criteria as established by Harding-Lawson Associates and are in conformance with preconstruction values used in the stability analysis. The recent Phase 5 construction was inspected by a Harding-Lawson engineer and Mr. Lewis reports that field density tests performed during construction show densities of at least 95% of optimum were achieved. Considering the low phreatic surface (as indicated by piezometers) compared to the phreatic surface used in the stability analysis, it is our judgment that Zonolite Tailings Dam conforms with the Recommended Guidelines stability criteria.

2.4 PROJECT OPERATION AND MAINTENANCE

Information on operations and maintenance was obtained from discussions with Mr. Robert Foss and Mr. Michael Ray, Chief Engineers for W.R. Grace and Company, Zonolite Operations. There is no formal operations and maintenance plan for the project.

2.4.1 Dam

The dam is located at the Zonolite mining operation site and is essentially under constant observation. Mr. Ray is responsible for the on-going maintenance and construction program for the dam and maintains a construction log (Reference 7) of all construction activity and data. Because the dam is being constructed in a phased program under the direction of an engineering design consultant, the embankment is essentially receiving major maintenance attention on a regular basis. Since 1971 when the starter dam was constructed, major construction modifications to the embankment have been performed every 2 to 3 years. Annual maintenance is also performed on the dam which includes dressing the slopes each spring or on an as-needed basis to repair minor erosional problems from surface runoff. Trees and brush are not allowed to become established on the embankment and burrowing animals are not of concern.

2.4.2 Reservoir

Zonolite Tailings Pond is a settling basin for the treatment of slurry mill tailings from the mining operation. The level of the pond is controlled by the decant tower and is maintained at an elevation such that the water from the surface meets necessary discharge quality requirements. As the sediments in the lake build up, storage volume and detention times are decreased and the pond's operating level is raised by placing stoplogs in the decant tower to meet water quality discharge requirements. Historically, the dam is raised when the normal reservoir operating level rises to within 10 to 12 feet of the dam's crest.

2.4.3 Warning Plan

There is no formal warning plan for use in the event of impending dam failure. However, because the project is occupied 24 hours a day early warning of unsafe conditions is probable.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspections of the dam, review of construction documents, and analysis of the project in terms of the recommended guidelines' performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification, and Safety Evaluation

The 135-foot-high Zonolite Tailings Dam could impound a maximum 2450 acre-feet of water with the pond at the crest of the dam and tailings at assumed elevation 2873.0 feet. Water storage decreases as tailings storage increases. In accordance with inspection guidelines, Zonoline Tailings Dam is large in size with a high downstream hazard potential rating. The recommended spillway design flood (SDF) for this project is 100 percent of the PMF. This dam safety study indicates that the project, with maximum water storage of 2450 acre-feet, controls a flood with hydrograph ordinates equal to approximately 45 percent of the PMF hydrograph ordinates. Larger floods will overtop the dam. The steady decrease of flood storage by tailings deposition will lower the flood routing capability. The dam is constructed of materials that would quickly erode and rapidly fail if overtopped by flood waters. Such failure could endanger life and property downstream. Because the project cannot safely handle the recommended SDF which is the full PMF, Zonolite Tailings Dam does not conform with inspection guideline hydrologic and hydraulic criteria.

3.1.2 Embankment

Zonolite Tailings Pond is impounded by an earthfill embankment dam. Since the construction of the starter dam in 1971 the embankment has been raised several times with the most recent being Phase 5. The additions to the embankment have been placed using downstream method of construction, i.e. the additional embankment is placed entirely on the downstream and crest portion of the dam which results in a downstream adjustment of the crest alignment.

The crest on the Phase 5 addition exhibits nearly a 2.0 feet range in elevation along its length. The range in elevation measured along a baseline, offset approximately 13 feet downstream of the centerline. The reason for the range in elevation is unknown as the dam is not equipped with settlement monitors and no "as built" design drawings are available. Annual inspections should monitor this condition. No visual signs of cracking, settlement, or slope instability were observed. Rill erosion on the embankment slopes was evident during both inspections. Seasonal maintenance of the slope erosion is required. Owner records and discussions with project personnel indicates embankment stability has not been a problem. There is no slope protection on the upstream slope of the dam, however, there is also no indication of wave erosion. Prevailing winds would be directed away from the embankment. The vertical distance from the normal reservoir level to the dam crest is adequate to prevent wind-generated waves from overtopping the embankment.

Drains placed in the embankment foundation have been extended to accommodate the additional fill from the phased construction activity. Only clear water has been observed seeping from the drain pipe and bedding material of the five perforated concrete drains. The drain nearest the northwest abutment which was dry in July 1980 was flowing in August 1981. In addition to the foundation drains, a 14-inch diameter steel pipe draining a spring in the foundation was discharging an estimated flow of 100 to 200 gpm. There was no evidence of a wetted front or seepage at the downstream toe of the embankment on July 25, 1980, however, during the August 13, 1981 inspection a wetted front equal in elevation to the flow line in the drains was observed.

A stability analysis for the proposed 200-foot high embankment was prepared in 1971 and is on file with the owner. The analysis was based on preconstruction data that have since been verified by field and laboratory test conducted during all phases of completed construction. Material strength results for all constructed phases of the dam are on file in the office of Harding-Lawson Associates. Piezometers have been installed and monitored with each phased addition to the embankment, however, attention to the changing tailings pond level should be noted with each reading.

Review of design stability analyses indicates that the stability analysis is adequate. In our judgement Zonolite Tailings Dam conforms with the Recommended Guidelines stability criteria.

3.1.3 Spillway and Reservoir Capacity

The reservoir has a surface area of about 68.5 acres and a water storage capacity of 2,120 acre-feet at the spillway crest, elevation 2,920 feet NGVD, assuming tailings to elevation 2873.0 feet. Approximately 330 acre-feet of surcharge storage is available between the elevation of the spillway and the crest of the dam. The discharge capacity of the spillway with the reservoir at the dam crest, is about 200 c.f.s. Because the structural integrity of the spillway is questionable, it is uncertain that the structure can safely contain the design flows without adversely affecting embankment safety.

3.1.4 Outlet Works/Decant Tower

The decant tower provides the normal operational releases from the tailings pond. Inspection of the Phase 5 tower and control facilities shows them to be in good operating condition. A welded steel decant line which rests on the pond floor and extends through the embankment is small and thus inaccessable for inspection without special equipment.

3.1.5 Operations and Maintenance

Zonolite Tailings Pond is operated as a settling basin for the milling operation at the mine site. Flow from Rainy Creek is diverted around the dam so under normal operating conditions the inflow to the pond is from the tailings discharge and from Fleetwood Creek. The pond elevation is regulated in small incremental rises to control the quality of discharges.

The embankment is essentially in a state of constant inspection and maintenance. Equipment necessary for dam maintenance is available at the mine site and maintenance is performed on an as-needed basis. There is no formal downstream warning plan for use in the event of impending dam failure.

3.2 RECOMMENDATIONS

Due to storage between normal pool and dam crest, the present project provides a degree of flood protection to the downstream area. The intent of report recommendations is to maintain or improve project safety, if feasible, without decreasing this flood protection.

The findings suggest that high priority be given the following recommendations:

1. Immediately develop, implement, and periodically test an emergency warning plan for use in the event of impending embankment overtopping or structural failure.

2. Periodically test the decant line in the section which passes through the embankment for possible leaks which could threaten the embankment. Repair if required.

3. Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard potential and to establish the safe minimum flood storage volume and spillway requirement. Studies should take into account the continually decreasing water storage volume as tailings are deposited and the effect of sudden release on the downstream environment. Monitor tailings accumulation and periodically evaluate the available flood storage volume. Evaluate the structural adequacy of the spillway under a full range of possible flow conditions. Remove the two tailings transfer pipes from the spillway approach section. Modify the project as studies indicate.

4. Continue to conduct inspections of the dam on an annual basis by engineers experienced in dam design and construction, continue to monitor and evaluate piezometers, foundation and toe drains and maintain a construction log of all additions and modifications to the project. Add piezometers during construction as required to define the phreatic surface in the dam. Any changes in the position of the phreatic surface should be fully evaluated with respect to its affect on stability. All existing and future piezometers should be sealed and capped to prevent tampering by vandals. Existing piezometers should be cleaned and unplugged or replaced if reliable and useful data cannot be collected.

Prior to performing engineering studies or remedial construction, coordinate with applicable State and Federal agencies to insure compliance with all pertinent laws and regulations.

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2. Fenneman, N.M., 1931, Physiography of Western United States: New York, McGraw-Hill Book Co., Inc., 534 p.

3. Johns, W.M., 1970 Geology and Mineral Deposits of Lincoln and Flathead Counties, Montana: Montana Bureau of Mines and Geology Bulletin 79, 182 p.

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8. U.S. Department of Army, Corps of Engineers Hydrologic Engineering Center; "HEC-2 Water Surface Profiles", Generalized Computer Program Users Manual with Supplement; Nov. 76.

9. U.S. Dept. of Agriculture - Soil Conservation Service, Average Annual Precipitation, Montana based on 1941-1970 base period, 13 pp, 1977.

10. U.S. Weather Bureau, Hydrometeorological Report No. 43, "Probable Maximum Precipitation, Northwest States," Washington, D.C., 1966.

11. Hydrometeorological Branch, U.S. Weather Bureau Office of Hydrology, "Memorandum" Probable Maximum Thunderstorm Rain for the Columbia River Basin East of the Cascade Ridge," 20 September 1967.

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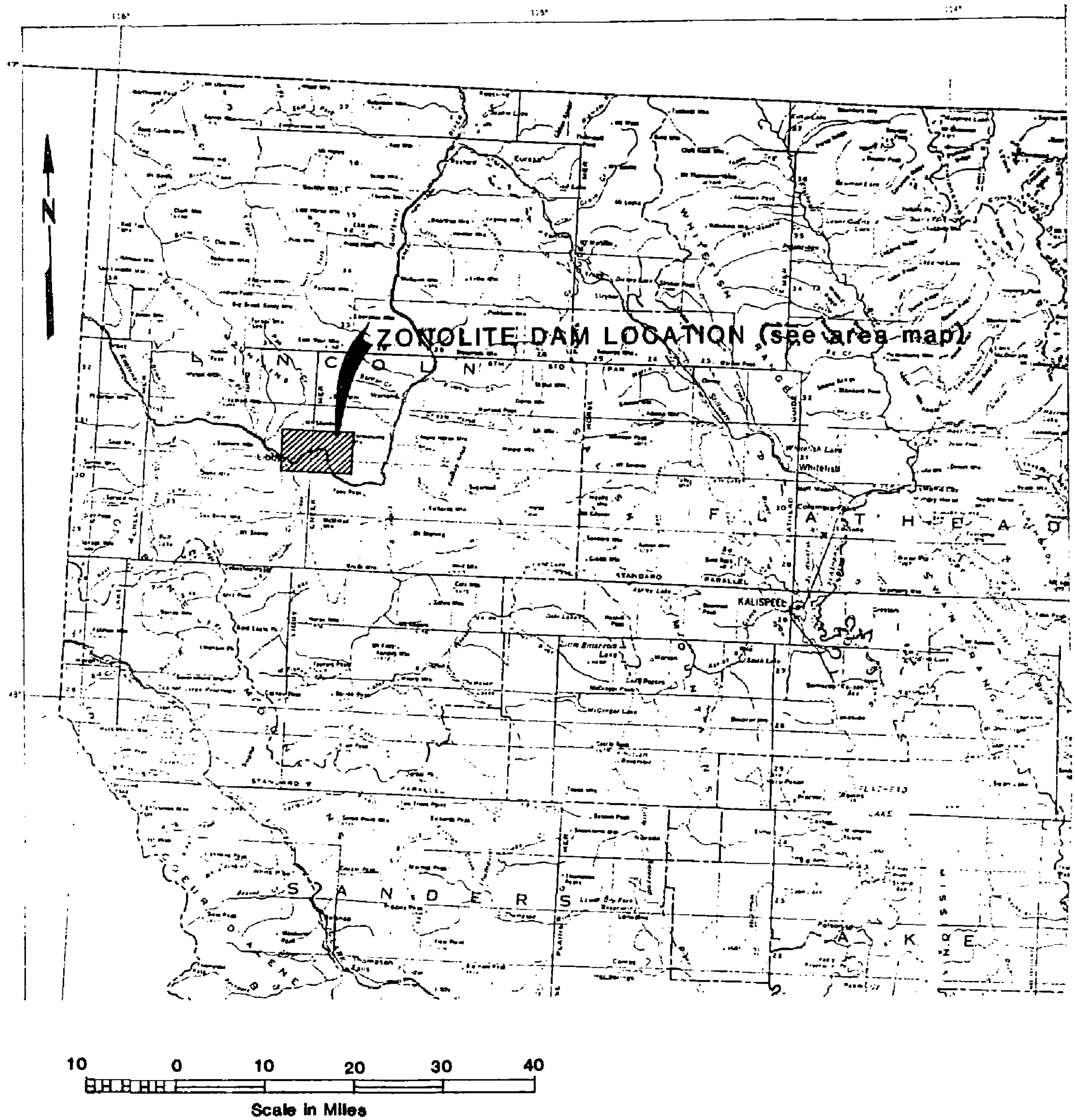
13. U.S. Army Corps of Engineers, Hydraulic Engineering Center, "Flood Hydrograph Package (HEC-1) Users Manual for Dam Safety Investigations", Davis, Calif., Department of the Army 1978, 93.

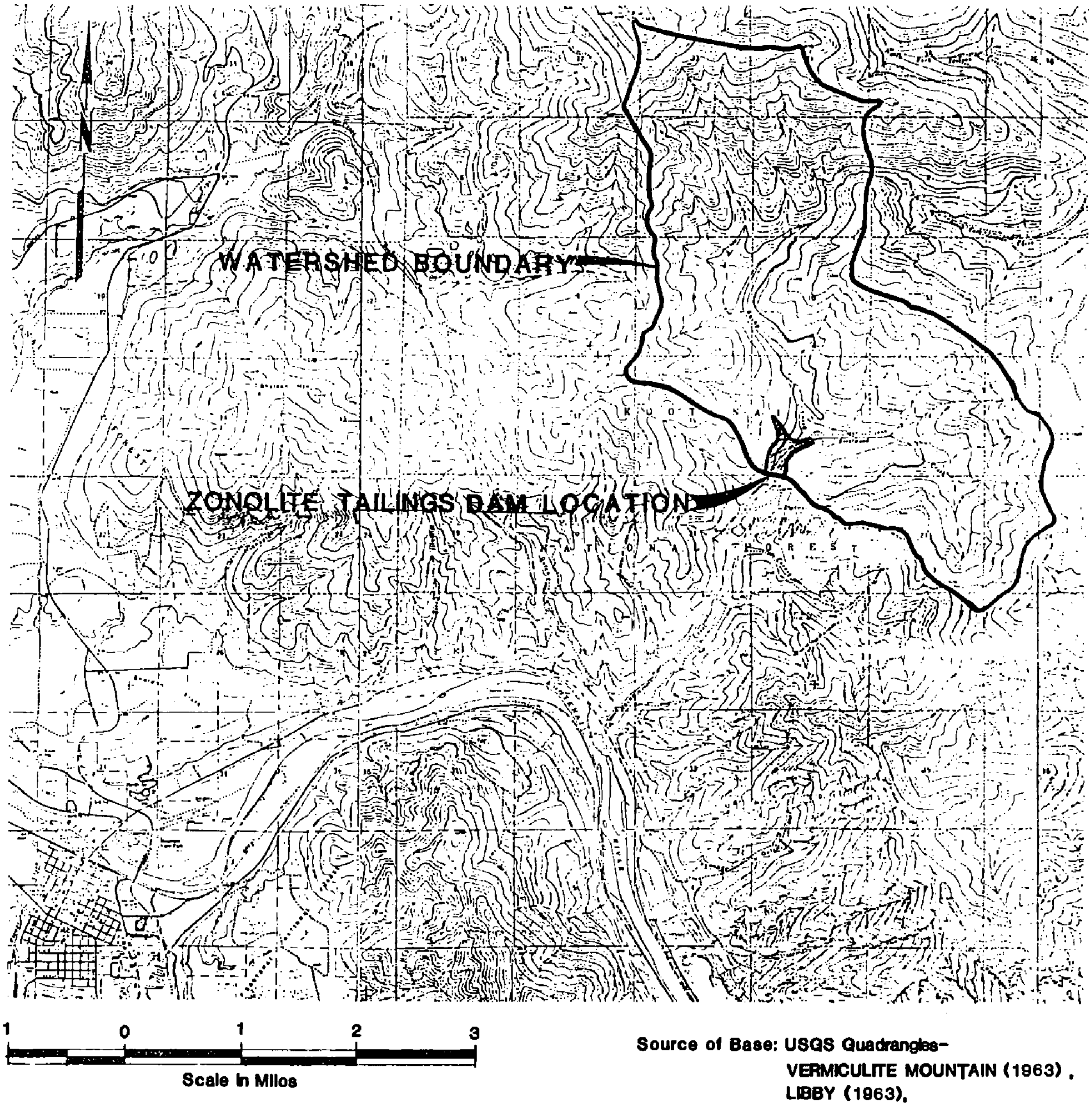
14. Lewis, L.E., 1971, Letter to Gene McKay, Bovay Engineers, Inc., re: Preliminary laboratory data, Tailings Dam, near Libby, Montana: Harding, Miller Lawson & Associates, January 21, 1971.

15. Lewis, L.E., 1971, Letter to Gene McKay, Bovay Engineers, Inc. Subject: Preliminary laboratory data, Tailings Dam, near Libby, Montana; with Particle Size Analysis, Compaction Test Data, Static Triaxial Shear Test Data, and Dynamic Shear Test Data: Harding, Miller, Lawson & Associates, January 21, 1971, 55 Mitchell Boulevard, San Rafael, California 94902.

16. Bovay, Engineers, Inc., 1971, Preliminary dam design and cost estimate for proposed tailings impoundment: for Construction Products Division, W.R. Grace & Co., Libby, Montana.

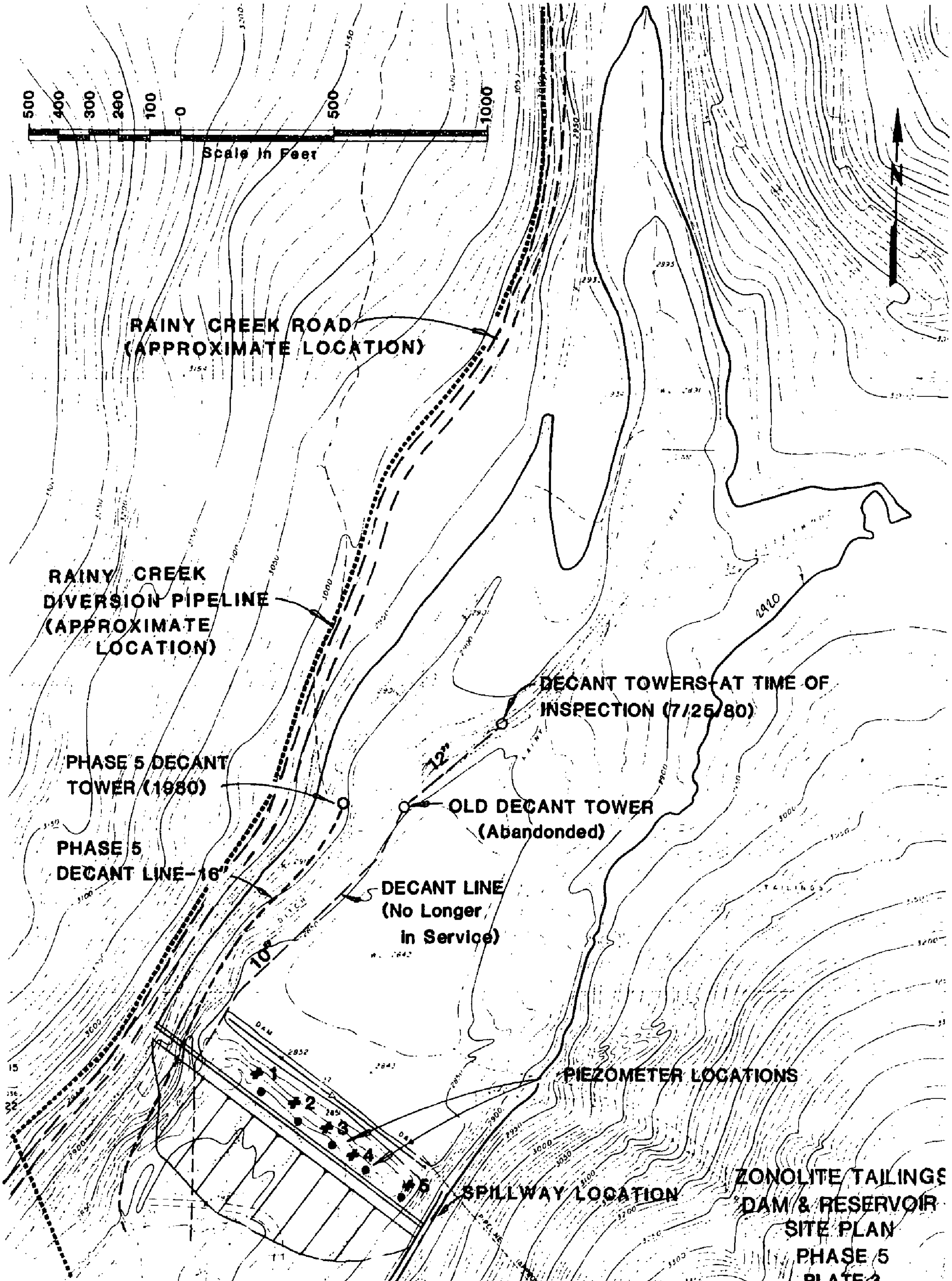


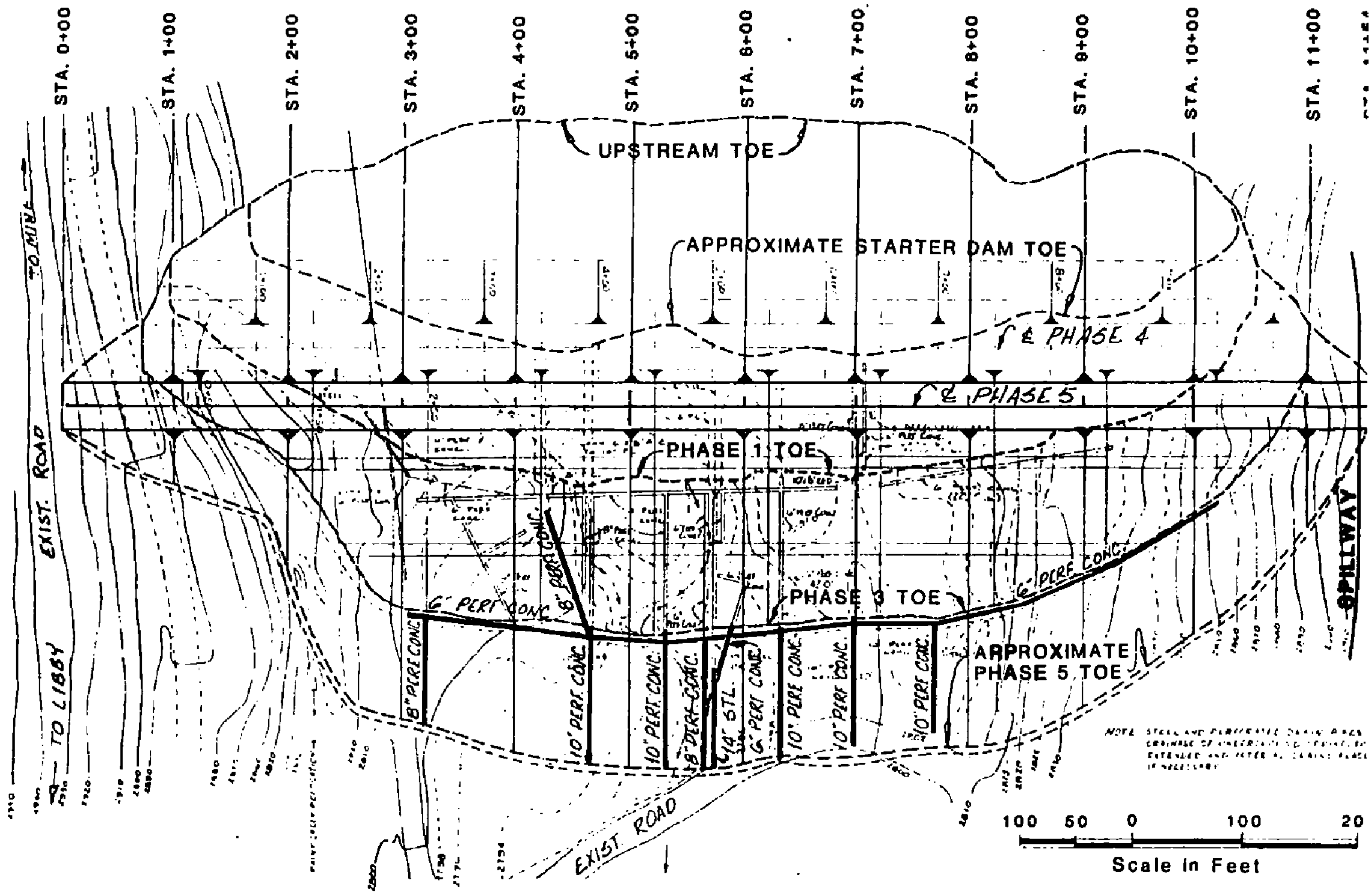
VICINTY MAP
PLATE 1



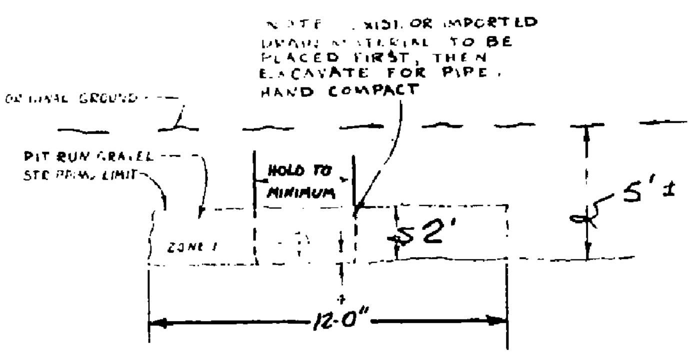
Source of Base: USGS Quadrangles-
VERMICULITE MOUNTAIN (1963),
LIBBY (1963),
ALEXANDER MOUNTAIN (1963)

ZONOLITE TAILINGS DAM
AREA MAP
PLATE 2


500 400 300 200 100 0 500 1000
Scale in Feet
N
RAINY CREEK ROAD
(APPROXIMATE LOCATION)
RAINY CREEK
DIVERSION PIPELINE
(APPROXIMATE
LOCATION)
DECANT TOWERS AT TIME OF
INSPECTION (7/25/80)
12"
PHASE 5 DECANT
TOWER (1980)
OLD DECANT TOWER
(Abandonded)
PHASE 5
DECANT LINE-16"
DECANT LINE
(No Longer
in Service)
10"
2920
PIEZOMETER LOCATIONS
#1
#2
#3
#4
#5
SPILLWAY LOCATION
ZONOLITE TAILINGS
DAM & RESERVOIR
SITE PLAN
PHASE 5
PLATE 3

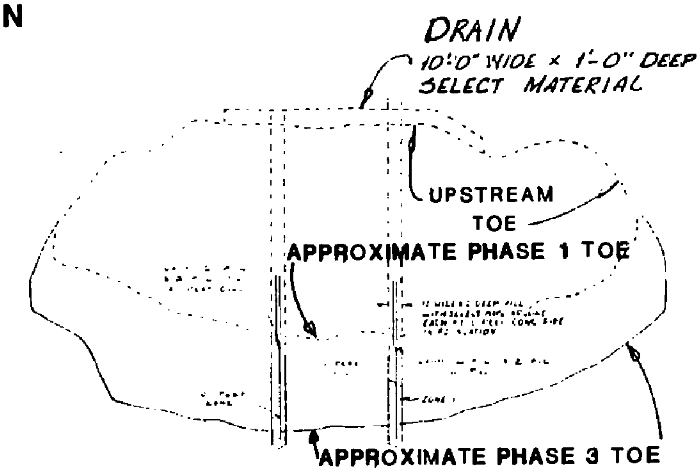


PLAN



TYPICAL DRAIN SECTION

NOT TO SCALE



TYPICAL FOUNDATION DRAIN PATTERN

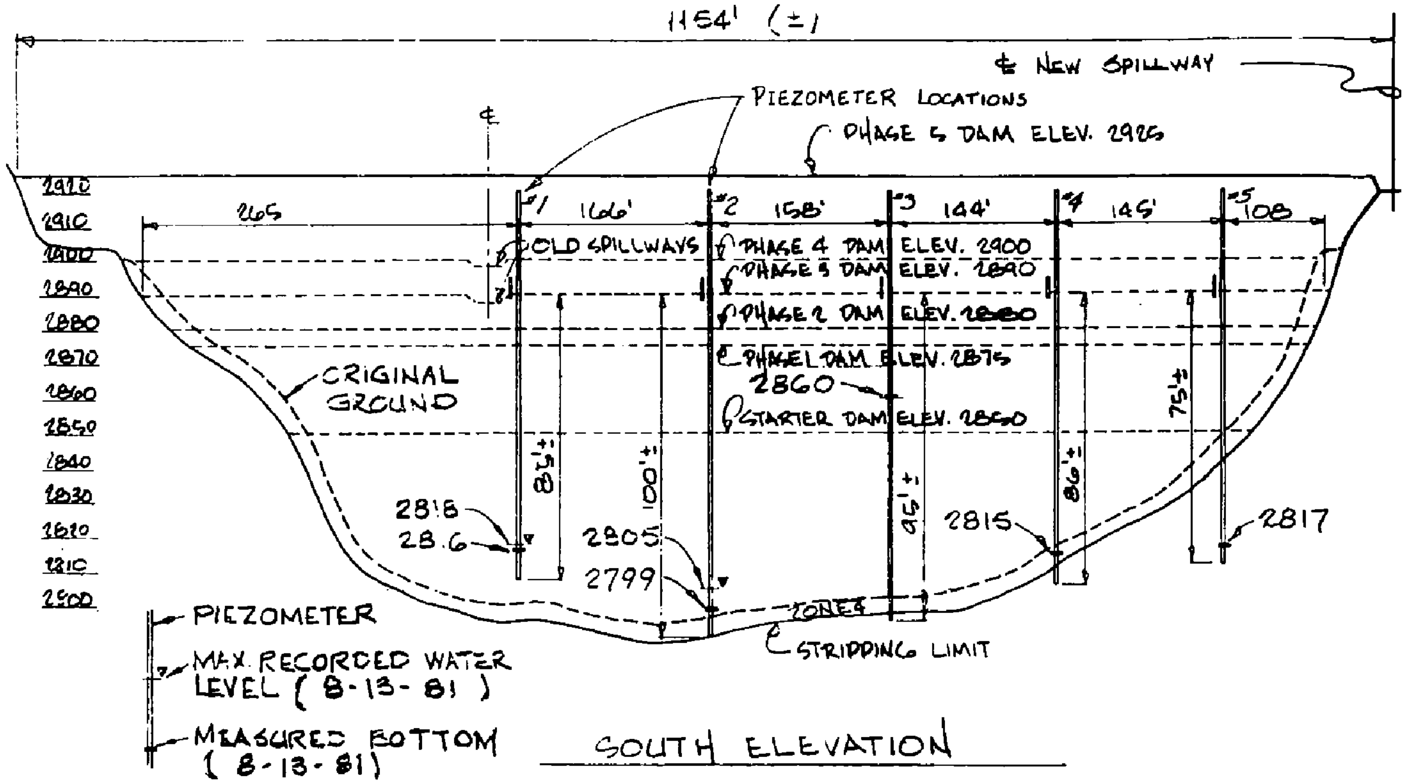
NOT TO SCALE

Source: W.R. GRACE & CO. Drawing No. 40-4009 4 (TAILINGS IMPOUNDMENT-PHASE 4)
Drawing No. 40-4009 4 (TAILINGS IMPOUNDMENT-PHASE 5)

ZONOLITE TAILINGS DAM

PLAN VIEW

PLATE 4



SOUTH ELEVATION

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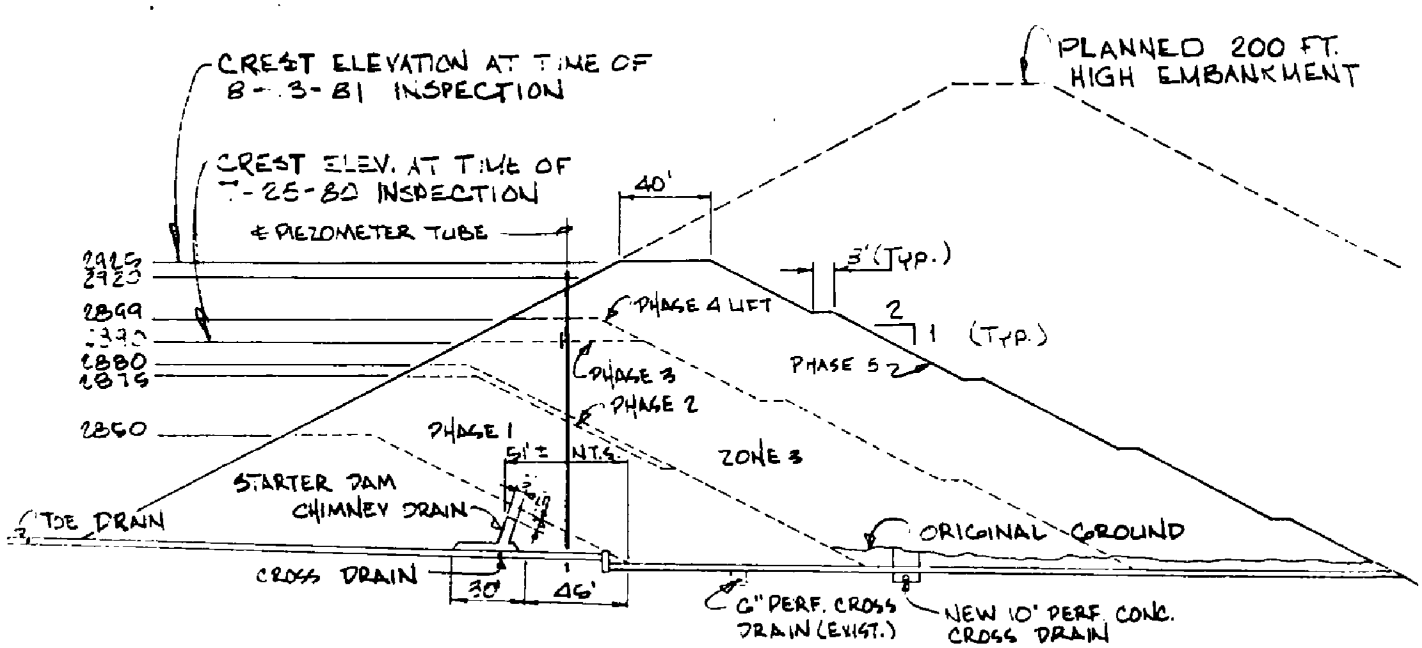


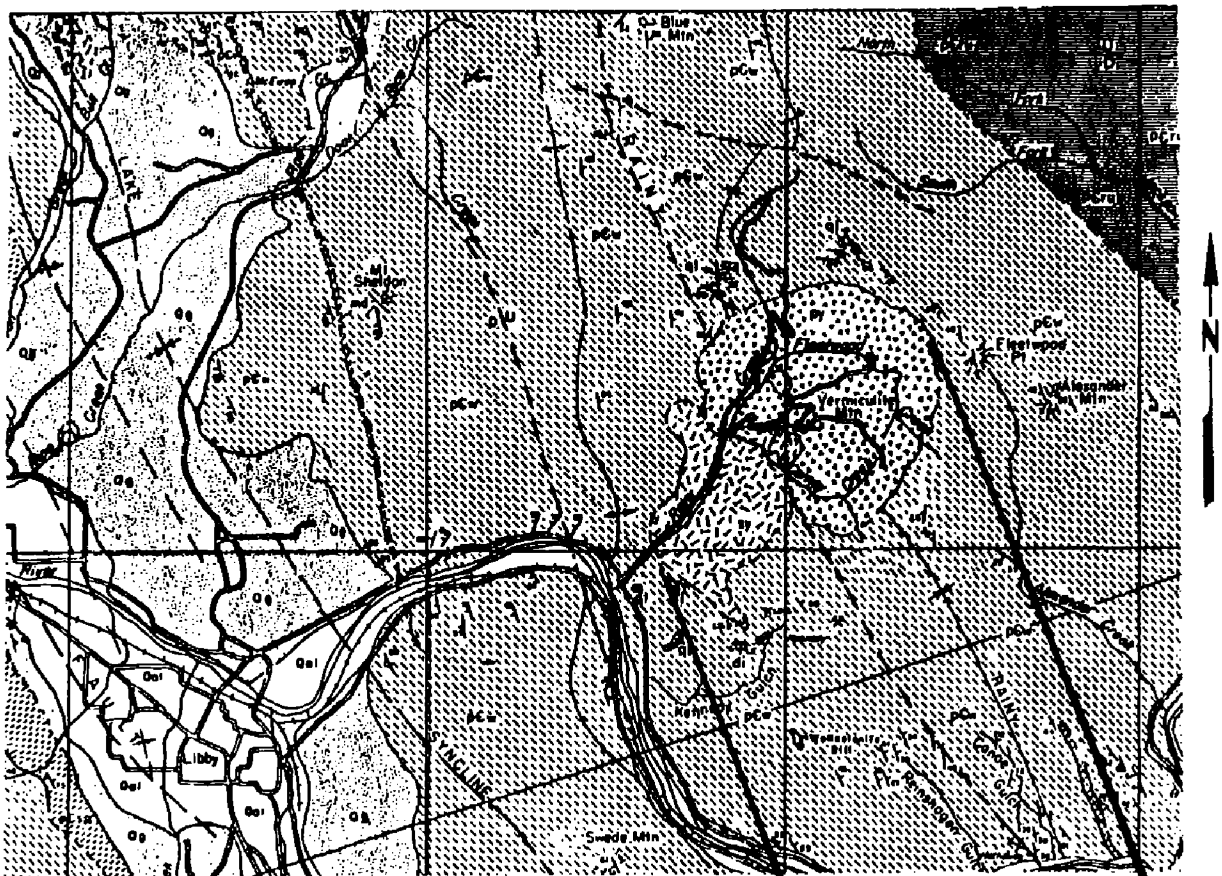
TYPICAL CROSS SECTION

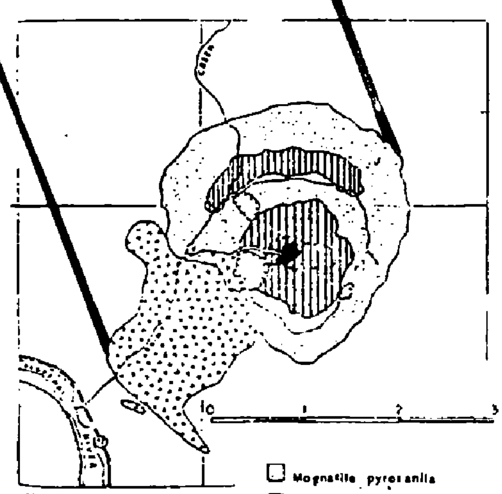
NOT TO SCALE

ZONOLITE TAILINGS DAM

PHASE 5 ADDITION

PLATE 5





Mine waste and mill tailings
Nepheline syenite
Syenite
Magnetite pyroxenite
Biotite
Pyroxenite
Bull Series

SEDIMENTARY ROCKS

pCsp

Striped Peak Formation

Gray and grayish-red argillite quartzite and gray-red-purple argillite. Some interbedded gray-green laminated argillite near center. Ripple marked and mud-cracked strata common, some casts of salt crystals.

pCru

Ravalli undifferentiated

Mapped as: medium- to light gray quartzite, quartzitic argillite, and argillite, some interbedded and laminated green-gray quartzite argillite weathering gray. In the Flathead Lake region grayish-red-purple, grayish-red, and grayish-green argillite and locally calcareous quartzite greenish-gray argillite. Upper Ravalli marker bed of calcareous argillite mapped as pCr[1].

pCw

Wallace Formation

Gray, green-gray, and yellow calcareous and dolomitic sandy argillite. discontinuous dolomite and magnesian limestone beds are common near the middle of the formation. Grayish-red shale and minor quartzite locally present near the top. Mud cracks and ripple marks common.

Qg

Quaternary glacial deposits

Undifferentiated lacustrine silt, clay, drift, gravel and alluvial fan material

Qal

Alluvium

Recent gravel, sand, and silt. Some Pleistocene glacial outwash

IGNEOUS ROCKS

Diorite dikes

Syenite stocks

Quartz latite porphyry dikes

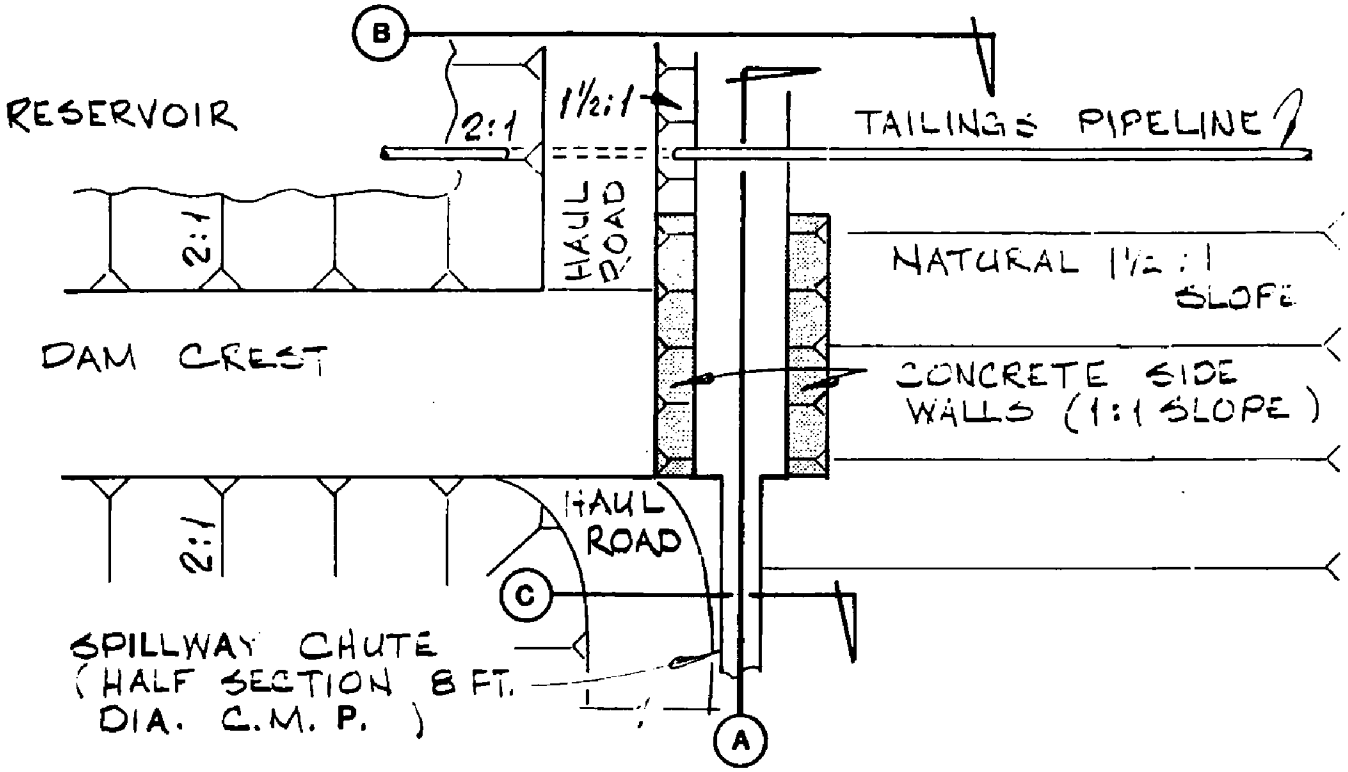
Basic stock composed of pyroxenite

mo

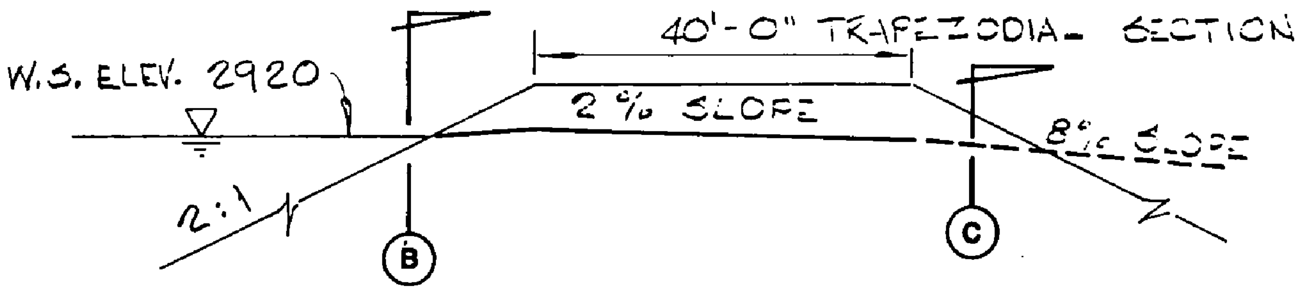
Metadiorite—dark-green gray and gray-black dikes sills, and stocks mapped by others as gabbro, hornblende gabbro, amphibolite, and metadiorite

GEOLOGY MAP

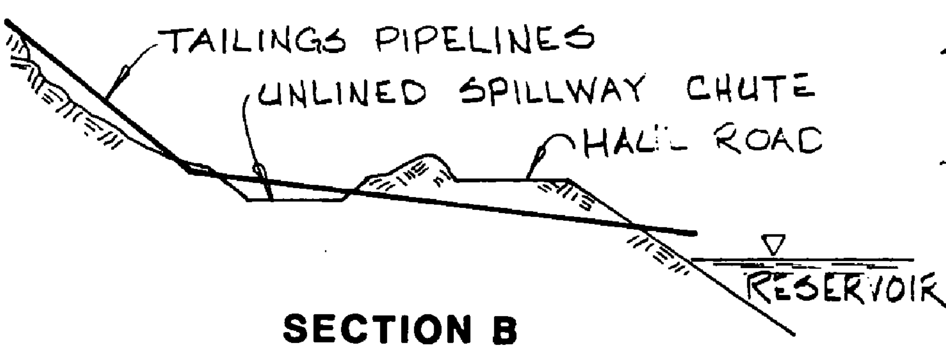
PLATE 6



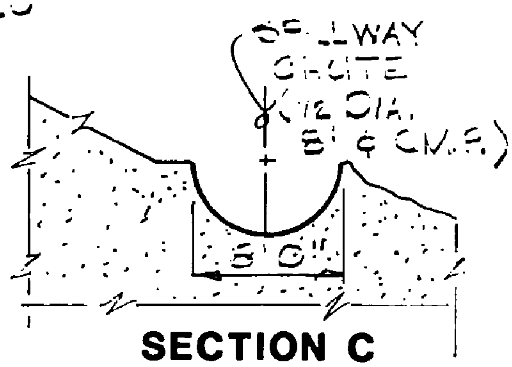
PLAN VIEW-SPILLWAY EAST ABUTMENT
SCALE: 1" = 40'



SECTION A-SPILLWAY PROFILE
SCALE: 1" = 20'

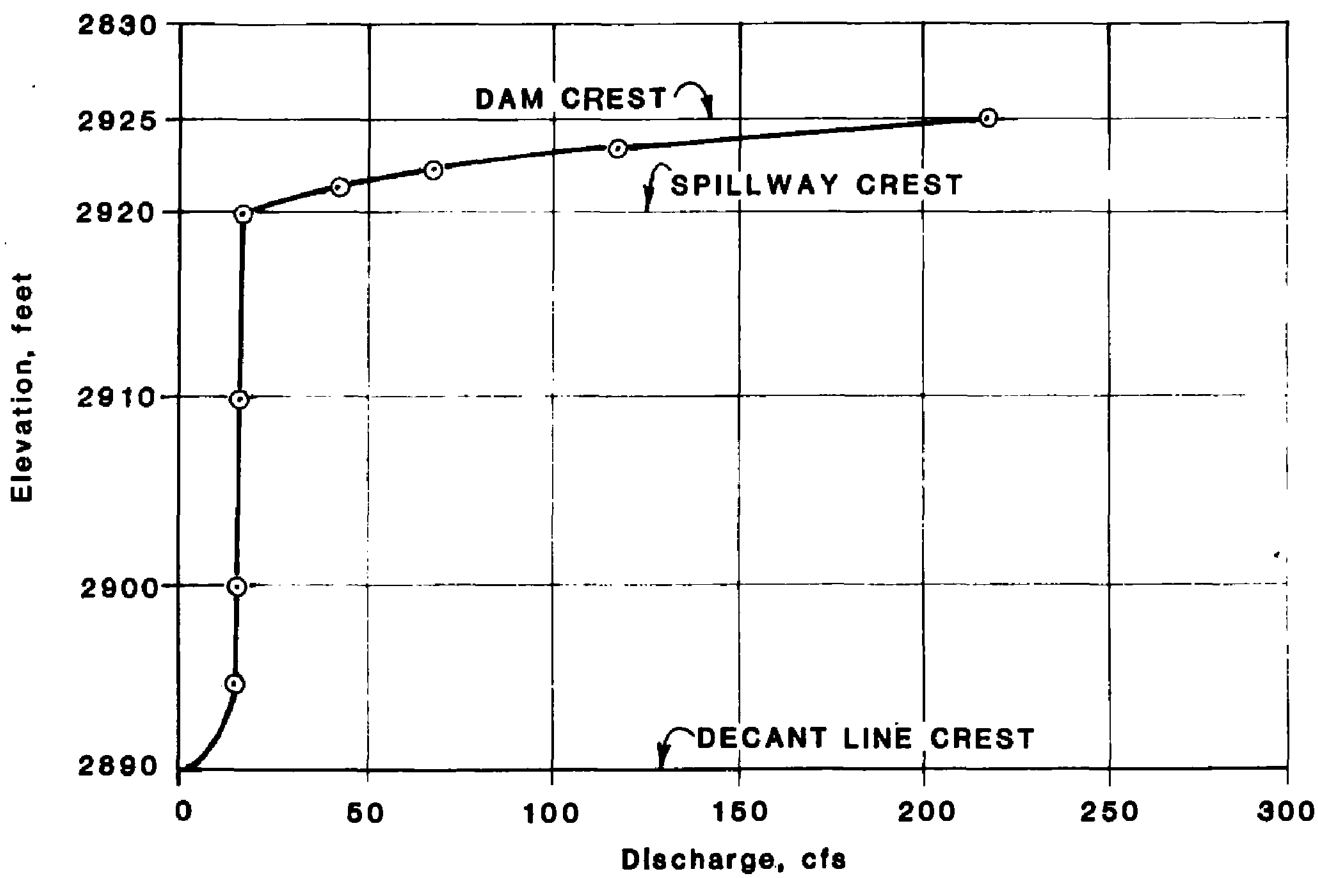


SECTION B
SCALE: 1" = 40'



SECTION C
NO SCALE

ZONOLITE TAILINGS DAM
SPILLWAY DESIGN
PHASE 5 TYPICAL SECTIONS
PLATE 7



ZONOLITE TAILINGS DAM
COMBINED DECANT LINE
&
SPILLWAY RATING CURVE
PLATE 8

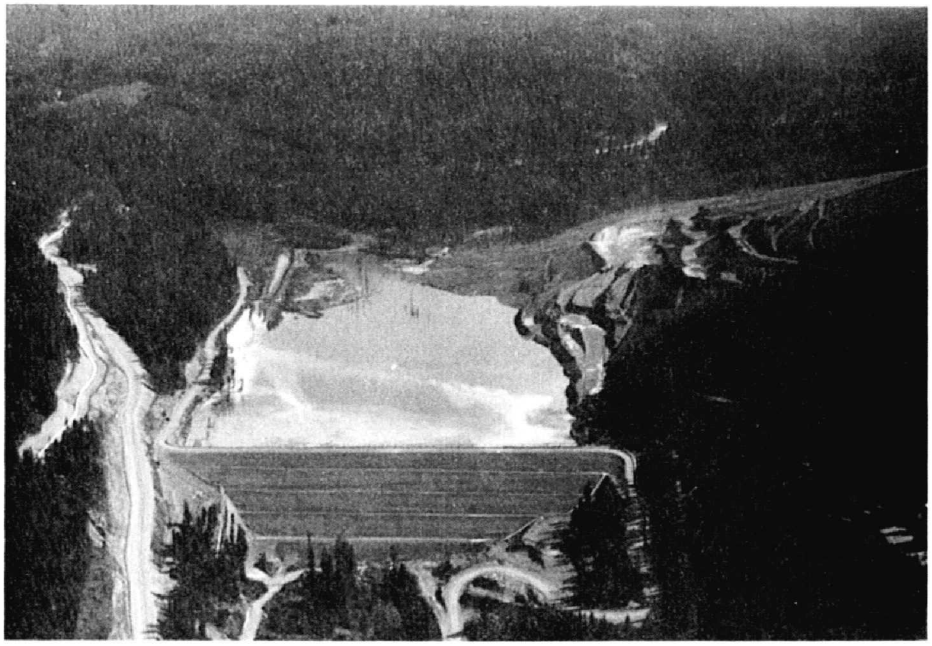


Photo 1   Aerial View of Zonolite Tailings Dam Looking Upstream

Photo 2   Aerial View of Zonolite Dam and Area Immediately Downstream (7-25-80)

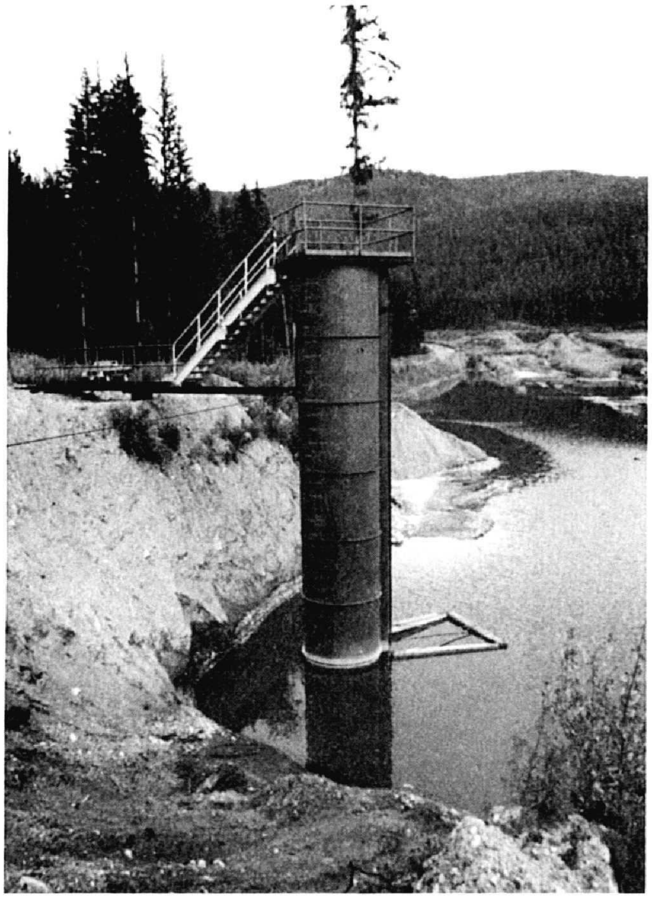


Photo 3 Phase 5 - Decant Tower
(7-81)

Photo 4

Rainy Creek Diversion Structure
Foreground: Entrance to Rainy Creek -
Background: Entrance to Diversion Pipeline
(8-13-81)

Photo 5 Rainy Creek Diversion Pipeline Along Rainy
Creek Road (7-81)

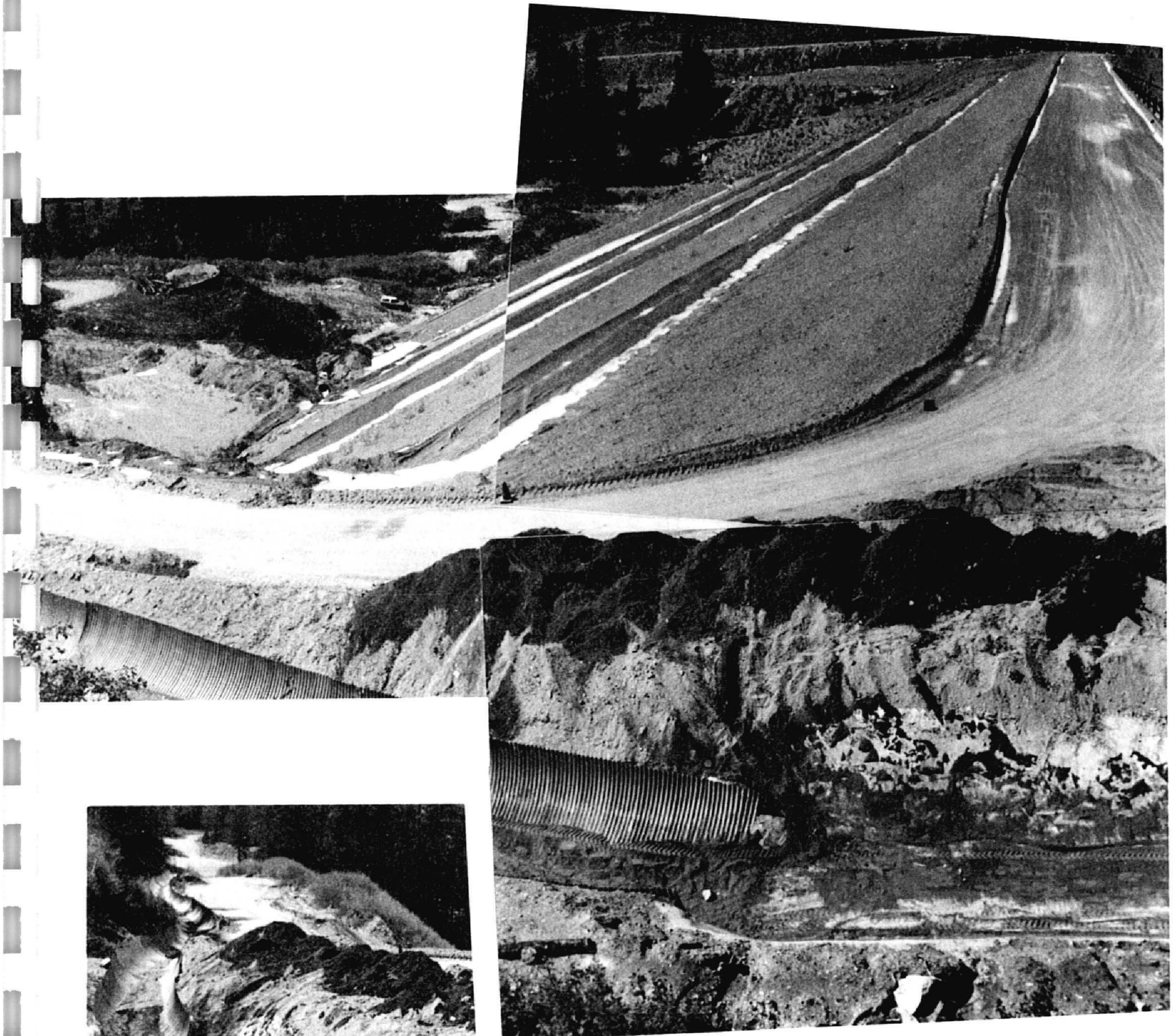


Photo 6 Zonolite Spillway and Downstream Face (8-13-81)

Photo 7 Spillway Chute and Approach Channel (8-13-81)

Photo 8 Spillway Approach Channel Looking Downstream (8-13-81)

Photo 9 Spillway Chute Looking Upstream Near Crest.
Note Overlapping Joints (7-81)

Photo 10 Spillway Foundation Collar (8-13-81)

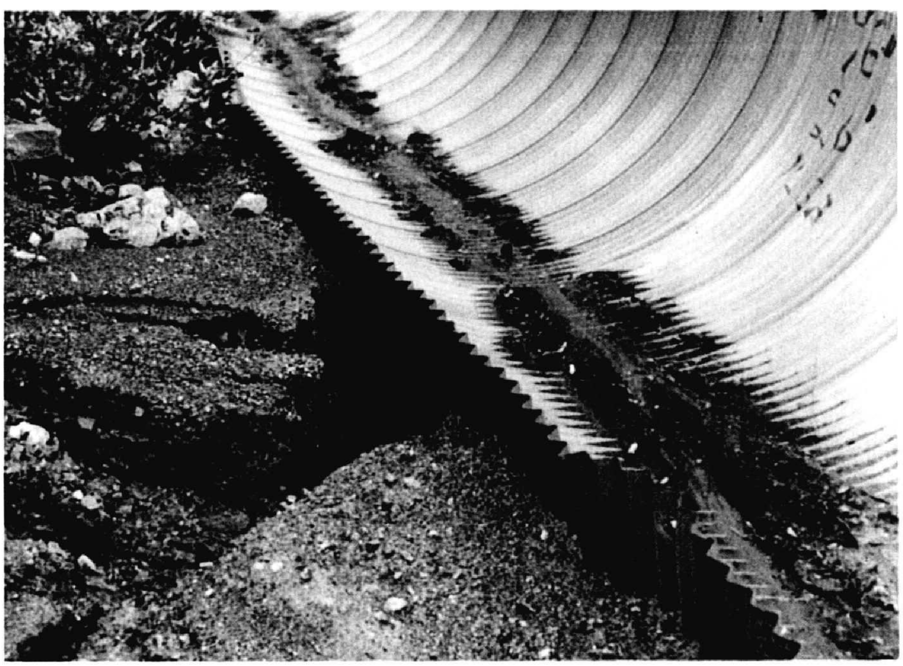


Photo 11 Erosion of Backfill and Undermining of Spillway Chute (7-81)

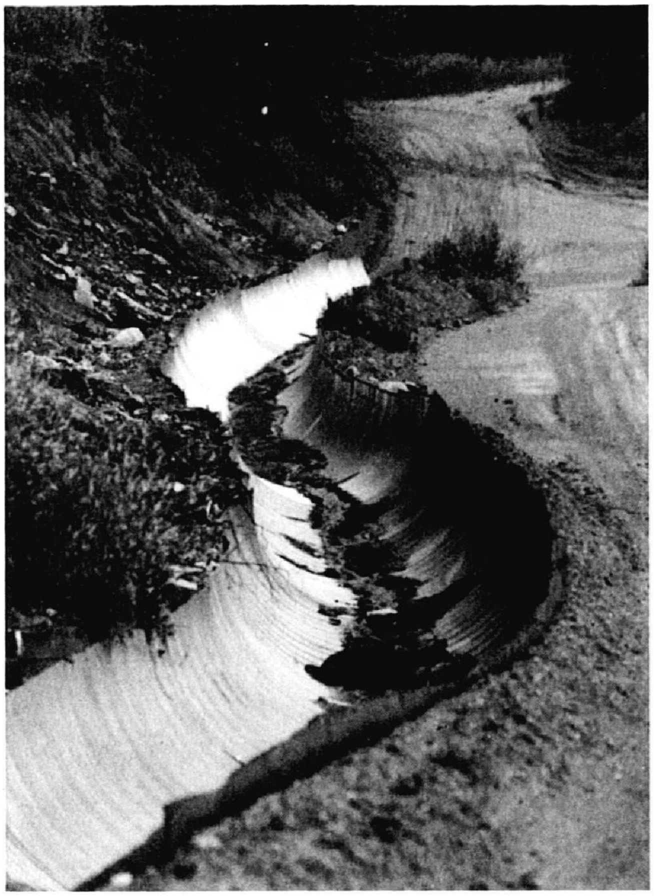


Photo 12 Bends in Spillway Chute Looking Downstream (7-81)

Photo 13 Downstream End of Spillway Chute Adjacent to Haul Road (8-13-81)

Photo 14 East Abutment Drain for Benches on Downstream Face (8-13-81)

Photo 15 West Abutment Area. Note Rill Erosion (8-13-81)

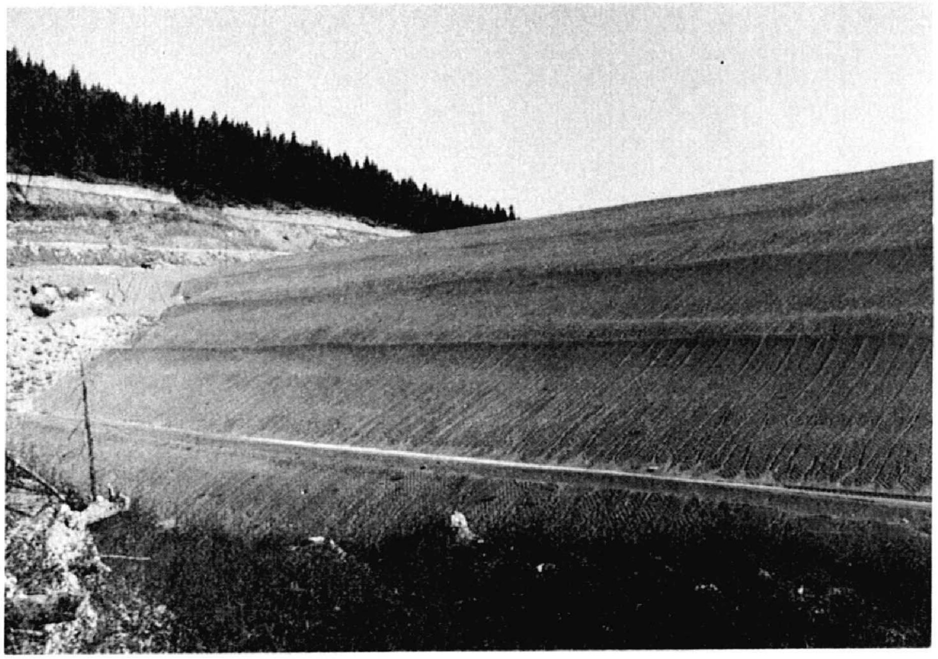


Photo 16 Downstream Face Looking Towards the West (8-13-81)

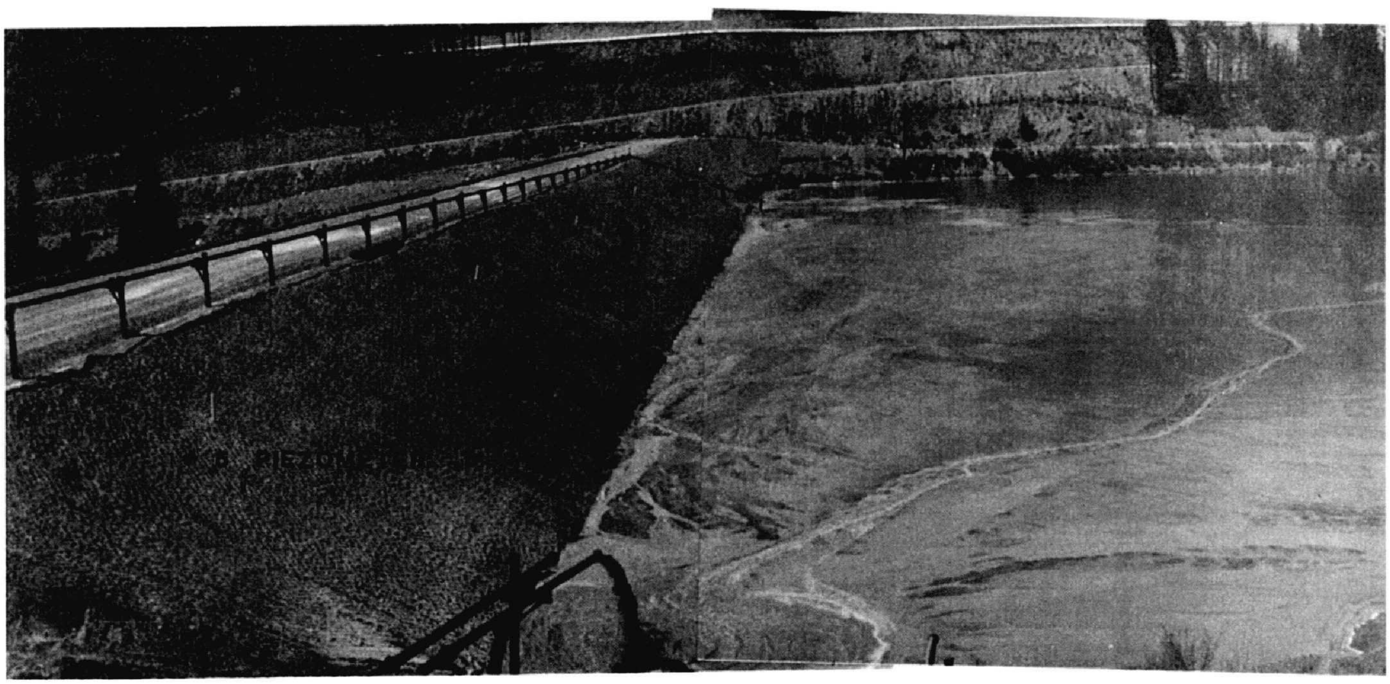


Photo 17 Upstream Face Looking Towards the West (8-13-81)

Photo 18 Zonolite Tailings Dam - Phase 5 - Crest
(8-13-81)

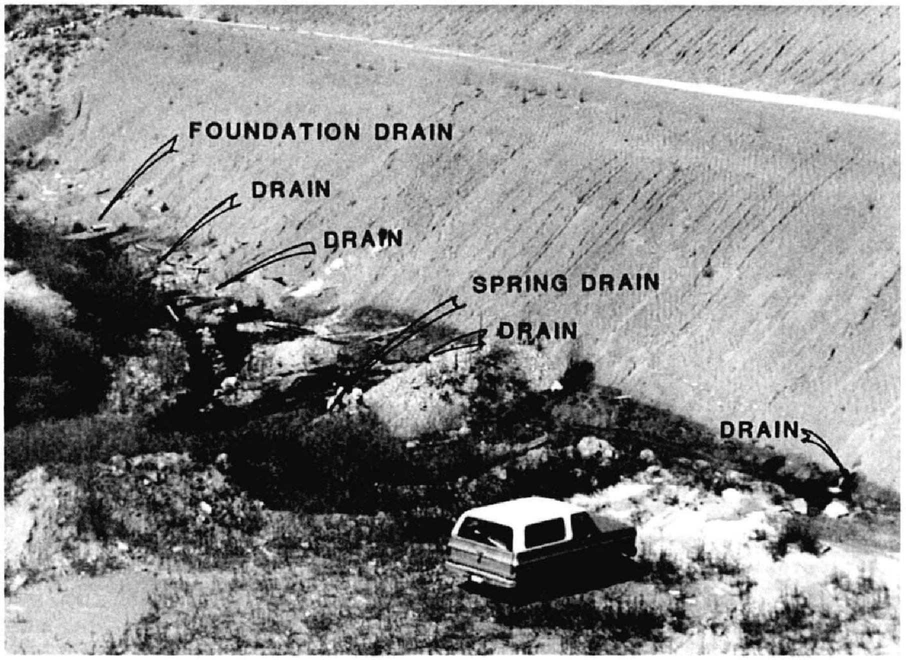


Photo 19 Foundation and Spring Drains. Note
Wetted Front (8-13-81)

Photo 20 First Exposed Drain Closest to East (left) Abutment (8-13-81)

Photo 21 Seepage from Bedding Material Around Spring Drain (8-13-81)

Photo 22 Foundation and Spring Drains Looking Towards the East (8-13-81)

Photo 23 Third Exposed Foundation Drain from East Abutment (8-13-81)

Photo 24 Fourth Exposed Foundation Drain from East Abutment (8-13-81)

Photo 25 Fifth Exposed Foundation Drain from East Abutment (8-13-81)

APPENDIX

Correspondence

GRACE

Zonolite
Construction Products Division

W.R. Grace & Co.
P.O. Box 609
Libby, MT 59923

(406) 293-4131

Sept 4, 1981

Dept. of the Army
Seattle District, Corp. of Engineers
P. O. Box C-3755
Seattle, Washington 98124

Attn: R. P. Sellevold, P.E.
Chief, Engineering Division

Gentlemen:

W. R. GRACE & CO. would like to offer the following as comments to the Phase I inspection report - National Dam Safety Program - Rainy Creek Basin Zonolite Tailings Dam, Libby, Montana MT-1470 Dated July 1981 Revised.

Executive Summary Comments:

I. The location of the dam is stated to endanger lives in the event of a failure. Actually there is no residential area between the dam and the Kootenai River. Danger downstream of the mouth of Rainy Creek would be minimal because the Kootenai River, with a maximum flood stage rating of 95,000 CPS and with Libby Dam acting as flood control, is adequate to handle the flows projected by this report. A dam failure could force evacuation of the W. R. Grace facilities near the mouth of Rainy Creek, but to state that such an event would endanger lives is not justified in fact.

II. With only 30 days allowed to prepare comments, it is impossible for W. R. GRACE to have a hydrologic and climatologic study prepared. However, the probable maximum flood (PMF) presented in this report must be questioned. In order to judge the rational behind the PMP, we would like to present the following comparisons:

a. The PMF generates a 43,400 cfs flow in the 9.7 square mile basin. This is 35.9% of the maximum flood on record in the Kootenai River (121,000 CFS at Libby in 1916)

b. Average uncontrolled flow in the Kootenai River for 52 years on record is July 1, 30,000 cfs; Aug. 1, 13,000 cfs and is Sept. 1, 17,000 cfs.

c. If in fact this flow rate occurred and if the Zonolite Tailings Dam had spillway capacity to handle it (or if the dam did not exist and Rainy Creek was in it's natural channel), the amount of damage done thru the relatively steep canyon and at the confluence with the Kootenai, would make the integrity of the dam immaterial. Parts of lower Rainy Creek cascade down a 12% grade. Damage in the canyon, at the highway crossing and to the W. R. GRACE facilities near the mouth of the creek, would be extensive if the dam withstood the storm or not.

d. The volume predicted at 3770 acre - feet is 1.78 times the calculated volume retained behind the dam.

III. Spillway capacity is indicated to be 200 cfs. Using standard open channel flow calculation techniques, the capacity of the trapazoidal spillway feeding the 1/2 round lined channel is 760 cfs. The calculations show the 1/2 round discharge channel governs capacity.

IV. The stability analysis has been made available to C of E representatives. The question of the adequacy of the spillway cannot be addressed without correcting the calculated capacity of the existing spillway and without flow requirements from a rational PMF.

V. Comments on the Recommendations.

a. Any such storm as predicted in the report would provide adequate warnings to anyone down stream and no formal downstream warning plan is needed.

b. Any decant line leak which would present a problem would be detected by visual inspections and/or show up as an otherwise unexplained rise in the water level in the closest piezometer well.

c. The Zonolite tailings dam is not intended to provide flood storage or protection.

d. The question of adequate spillway capacity cannot be answered without reconsideration of design conditions.

e. W. R. GRACE has and intends to continue using competent engineers to monitor the condition of the dam.

Zonolite
Construction Products Division

GRACE

W. R. GRACE & CO. appreciates the opportunity to comment on this report prior to publication. The magnitude of the probable maximum flood used in this report is questionable and the conclusions and recommendations based on it must be reconsidered based upon any changes.

Very truly yours,

CONSTRUCTION PRODUCTS DIVISION
W. R. GRACE & CO.

William J. McCaig

W. J. McCaig
General Manager
Libby Operations

WJM/ns

cc: J. Wolter, M. Ray

DEPARTMENT OF NATURAL RESOURCES AND CONSERVATION

WATER RESOURCES DIVISION

TED SCHWINDEN GOVERNOR

32 SOUTH EWING

STATE OF MONTANA

(406) 449-2072 ADMINISTRATOR
(406) 449-3962 WATER RIGHTS BUREAU
(406) 449-2872 WATER SCIENCES BUREAU
(406) 449-2864 ENGINEERING BUREAU
(406) 449-2872 WATER PLANNING BUREAU

HELENA, MONTANA 59620

September 10, 1981

Department of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, Washington 98124

Attention: Ralph Morrison

Dear Ralph:

Re: Morrison-Maierle, Inc. Dam Safety Inspection Report of Zonolite Tailings Dam MT-1470.

We have reviewed the above referenced final draft report. We concur with the findings and recommendations and find that it satisfies the criteria of Phase I report.

Minor editorial comments have been discussed with your staff, and we understand these will be incorporated in the final report.

Thank you for this opportunity to review and comment on the final draft report on Zonolite Tailings Dam.

Sincerely,

Richard L. Bondy

Richard L. Bondy, P.E.
Chief, Engineering Bureau

RB:AT:lz

U. S. Department of Labor

Mine Safety and Health Administration
P O Box 25367
Denver, Colorado 80225

SAFETY AND HEALTH TECHNOLOGY CENTER
Mine Waste and Construction Division

September 1, 1981

Report No. D3674-W1499
File: HLS-5

MEMORANDUM FOR: WILLIAM C. GARDNER
District Manager, Rocky Mountala District
Metal and Nonmetal Mine Safety and Health

FROM: John L. Odell
JOHN L. ODELL
Acting Chief, Mine Waste and Construction Division

SUBJECT: Review of Phase I Inspection Report, National Dam Safety Program by Morrison-Maierle, Inc. for the Zonolite Tailings Dam, near Libby, Lincoln County, Montana, Construction Products Division of W. R. Grace & Company

In response to a request by Wilbur Guthrie, Jr., Supervisory Mine Inspector, Salt Lake City Subdistrict Field Office, Helena, Montana, the Phase I Inspection Report, National Dam Safety Program, prepared by Morrison-Maierle, Inc., Consulting Engineers, was reviewed by the Safety and Health Technology Center, Denver, Colorado. The Seattle District Corps of Engineers has requested MSHA critique the report and submit their comments. The report was evaluated for compliance with MSHA's design guidelines for impounding structures associated with metal/nonmetal mines and MSHA's current regulatory standard, section 57.20-10, title 30, Code of Federal Regulations, which states, "if failure of a water or silt retaining dam will create a hazard. It shall be of substantial construction and inspected at regular intervals."

According to the report, the dam is located such that its failure could endanger many lives and cause excessive economic loss. The report also indicates that the hydraulic facilities can handle up to 45 percent of the probable maximum flood (PMF). Larger floods would therefore overtop the dam. Since failure can cause loss of life, this office recommends that the facilltes be capable of handling 100 percent of the PMF.

The reservoir routing mentioned in the report was started at the minimum opening elevation of the decant tower. Since stoplogs will be placed in the decant opening as the tailings rise in the impoundment, it would appear prudent to begin the flood routing at the top elevation of the final stoplog.

Two pipes cross the spillway approach channel at an elevation only slightly above the channel invert. The pipes and any debris trapped by the pipes can significantly reduce flow through the approach channel. Each pipe should be relocated to eliminate any impediment of flow.

it appears that a portion of the approach channel sideslopes is formed by embankment material of the dam. These slopes should be properly lined to prevent erosion of embankment material.

The spillway chute, consisting of a half section of 8-foot diameter corrugated metal pipe and sloped at a relatively steep 8 percent, is only anchored at the dam crest. We concur with the findings stated in the report that the structural integrity of the spillway chute would be questionable when carrying design flows. Since a failure of the chute may affect embankment safety, revisions in design and construction should be made.

Although the report states that the computed safety factors. in regard to embankment stability, exceed minimum recommended allowable safety factors for both static and seismic loading, the safety factors obtained were not stated. Our guidelines require minimum static and seismic factors of safety of 1.5 and 1.2 respectively, under maximum normal anticipated phreatic conditions.

Plate 5 indicates that two piezometers are located on the upstream face of the phase 5 embankment construction. Although the piezometers would have been beneficial in locating the phreatic line during the initial phases of embankment construction, the piezometers during phase 5 will be too close to the water surface in the impoundment to provide any significant data.

it appears that the total stress method was used in determining shear strengths for the stability analysis. For long-term design, an effective stress analysis would have been more appropriate. Also, a dry density of 138 pounds per cubic foot and cohesion of 4000 pounds per square foot seems to be unusually high for gravelly sand material, especially the cohesion in a consolidated-undrained triaxial test. in essence, an embankment with a downstream slope of 2 horizontal to 1 vertical and constructed to the height indicated in the report can be very marginal in regard to obtaining a 1.5 static factor of safety. Therefore, this area of the investigation should be emphasized strongly to verify the designs.

The drawings and most of the report indicate the embankment is ultimately to be 125 feet high. it is assumed that the embankment will be limited to 125 feet rather than the planned 200 feet indicated on pages vii, 10, 11, and 29 of the report.

if we can be of further assistance, please let us know.

cc: T. Shepich
J. Mulhern
F. Delimba
W. Guthrie
R. P. Sellevold

DEPARTMENT OF THE ARMY
SEATTLE DISTRICT. CORPS OF ENGINEERS
P.O. BOX C-3755
SEATTLE, WASHINGTON 98124

NPSEN-FM

10 SEP 1981

Mr. William McCalg
W. R. Grace and Company
P.O. Box 609
Libby, Montana 59923

Dear Mr. McCaig:

Thank you for your comments regarding the Phase I inspection report on Zonolite Taillngs Dam. While your comments will be considered in preparation of the final report, we will further address some of your noted concerns.

As the report indicates, the evaluation of the hazard potential is based on engineering judgement and is not supported by a detailed study and/or dam breach analysis. If, as you suggest, the hazard was down-graded from "high" to "significant," the inspection guidelines would still recommend the project (because of its size) be capable of handling the full PMF.

By definition, the PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in a given region. The probable maximum precipitation for your area was obtained from data published by the U.S. Weather Bureau. We have recommended that more detailed studies be performed to more clearly define the downstream hazard and appropriate spillway design flood along with the recommended minimum water storage volume. The outflow capacity for the spillway is governed by the entrance to the half-round pipe spillway regardless of the approach channel capacity. The control section has a maximum flow capacity of 200 cfs. This outflow was used in the PMF routing.

Thank you for your assistance and cooperation in inspecting Zonolite Tailings Dam.

Sincerely,

R.P. Sellevold

R.P. SELLEVOLD, P.E.
Chief, Engineering Division

MORRISON · MAIERLE, INC

QUALITY ASSURANCE

QA

Roger C Foster

Project Manager

Thomas M Watson

Branch Manager or ~~Department Head~~

Philip P. Porrini

Peer Reviewer

C.W. Keith

Principal - In - Charge

Chief Engineer No.	Date Approved
81 - 17	9 - 18 - 81

Project No 1447-09-03 (33)

DEPARTMENT OF THE ARMY
SEATTLE DISTRICT. CORPS OF ENGINEERS
P.O. BOX C-3755
SEATTLE. WASHINGTON 98124

NPSEN-FM

Mr. Rick Bondy
Chief, Engineering Bureau
Montana Department of Natural
  Resources and Conservation
32 South Ewing
Helena, Montana 59601

Dear Mr. Bondy:

Inclosed are 15 copies of the approved dam safety inspection report on Zonolite Tailings Dam, prepared in accordance with Public Law 92-367, 8 August 1972.

This report presents an executive summary of the project, background Information, details of the inspection and records evaluation, findings and recommendations.

Public release of the inspection report and initiation of public statements fall within the Governor's prerogatives. In addition to any public release the Governor might make, the U.S. Army Corps of Engineers will respond to news media and citizen inquiries and make the report accessible on request.

Sincerely,

LEON K. MORASKI
Colonel, Corps of Engineers
District Engineer

1 Incl (15 copies)
As stated

A3—Reference 2

Schafer & Associates
P.O. Box 6186
Bozeman, MT 59715
(406) 587-3478

Waste Management
Land Reclamation
Resource Inventory
Agricultural Consulting

July 22, 1991

RECEIVED
JUL 2 3 1992
STATE LANDS

Mr. Pat Flantenberg
Department of State Lands
Hardrock Mining Bureau
1625 Eleventh Avenue
Helena, Montana 59620

Dear Pat:

Enclosed is a copy of our calculations of the rain on snow PMF event related to the proposed flood routing design at the W. R. Grace vermiculite mine. This should help you address this question in your Environmental Assessment report. We used a veiy conservative approach assuming an unlimited supply of snowpack, snowpack coverage over the entire drainage, and no allowance for infiltration losses. The January storm appears to provide the highest possibility for producing the rain on snow PMF. We calculated a 3-day storm of 11.1 inches producing an additional 2.8 inches of water in snowmelt. This storm was preceded by a 3-day pre-storm melt period which produced a rather substantial runoff of 140 cfs. This volume is successfully routed through the proposed tailings impoundment design without loss of storage capacity.

The PMP produced a peak discharge of 3704 cfs, substantially less than the 11,676 cfs produced in the thunderstorm PMP. However, this comparison is deceiving. Without an emergency spillway, the 4 ft. by 8 ft. box culvert safely passes a 0.50 PMF thunderstorm event while it will pass a 0.53 PMF rain on snow 3-day general storm PMF. Thus, the two storms a really quite comparable in terms of impoundment surge capacity. The proposed emergency spillway will provide additional flood routing capacity in both cases.

The Forest Service inquired about the .5 PMP recorded in the area which was referred to in our report. That event occurred on November 21-22, 1909 and rainfall data was taken at Snowshoe, Montana; 7.0 inches of precipitation was recorded in a 48 hour period. The same storm produced similar events in Rattlesnake Creek, Idaho and Sheep Hill, Idaho. This data is recorded in Hydrometeorological Report No. 43, Probable Maximum Precipitation, Northwest States (Tables 7-2 and 7-3).

If you have any questions or need additional information please call.

Sincerely,

Tom Hudson

Tom Hudson
Project Manager

CALCULATION OF RAIN ON SNOW PMF FOR THE PROPOSED TAILINGS IMPOUNDMENT STREAM ROUTING AT THE W.R. GRACE MINE, LIBBY MT.

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Schafer and Associates

Project: W R CAGE
Client Number: 102 Name: TAILINGS ALGORNA
Date: 7/17/92
Sheet 1 of
By: KH

CALCULATION OF PMF FOR
A RAIN ON SNOW PMP EVENT

I PMP

THE PROCEDURE USED FOR CALCULATION OF A WINTER GENERAL STORM PMP IS GIVEN IN CHAPTER VI OF HYDROMETEOROLOGICAL REPORT NO. 43 "PROBABLE MAXIMUM PRECIPITATION, NORTHWEST STATES" U.S. WEATHER BUREAU, NOVEMBER 1966.

CALCULATION WORKSHEETS TO DETERMINE THE 3-DAY GENERAL STORM FOR THE MONTHS OCTOBER THROUGH JUNE ARE ATTACHED. ALTHOUGH PMP'S FOR OCT-DEC AND APR-JUN ARE SOMEWHAT LARGER THEY ARE PROBABLY NOT SUITABLE CASES BECAUSE SNOWCOVER AT THE 3000-6000 FOOT ELEVATION IS LIKELY TO BE INCOMPLETE OR SNOWPACK MAY BE INADEQUATE. THE INTENT IS TO LOOK AT A SITUATION WHERE FROZEN GROUND OR ICE MAY EFFECTIVELY REDUCE INFILTRATION LOSSES. OF THE MONTHS JAN-MAR, JANUARY APPEARS TO PROVIDE THE BEST OPPORTUNITY FOR PRODUCING A PMP EVENT OF THIS KIND. THE MAXIMUM SUSTAINABLE DEWPOINT (WHICH DRIVES THE PMP CALCULATION) IS COMPARABLE TO THAT IN MARCH. THE SUSTAINABLE WINDS WHICH IMPACT SNOWMELT ARE ALSO HIGHEST IN JANUARY. CONSEQUENTLY A JANUARY 3-DAY GENERAL STORM HAS BE SELECTED FOR ANALYSIS.

II SNOWMELT

"SNOW HYDROLOGY", U.S. CORPS OF ENGINEERS 1956 PROVIDES METHODOLOGY FOR ESTIMATING SNOWMELT DUE TO CONVECTIVE HEAT EXCHANGE IN EQUATION 6-18b, WHICH APPLIES TO FORESTED AREAS:

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Schafer and Associates

Project: W.R. GRACE
Client Number: [illegible] Name: [illegible]
Date: 7/17/9[illegible] Sheet 3 of
By: TJH

$$M_A = K(0.0084v)(0.22T_a' + 0.78T_d') + 0.029T_a'$$

OTHER FORMULAE APPLY IN HEAVILY FORESTED, PARTLY FORESTED AND OPEN AREAS. THE VALUE IS IN INCHES/DAY

K = BASIN CONVECTION - CONDENSATION MELT FACTOR (A CORRECTION FOR RELATIVE WIND EXPOSURE)

V = WIND VELOCITY (mi/hr)

T_a' : TEMPERATURE DIFFERENTIAL BETWEEN AIR @ 10' AND THE SURFACE

T_d' : DIFFERENTIAL DEWPOINT BETWEEN AIR @ 10' AND THE SURFACE

A K VALUE OF 0.6 WILL BE USED WHICH IS INTERMEDIATE BETWEEN AN OPEN AREA (K=1) AND A HEAVILY FORESTED AREA (K=0.2)

V, T_a' AND T_d' WILL BE DETERMINED LATER BY METHODS IN HMR 43, CHAPTER VIII.

RAINFALL PRODUCES ADDITIONAL SNOWMELT AS RAIN IS COOLED TO 32° IN CONTACT WITH SNOW/ICE. QUANTITY IS GIVEN BY

$$M_R = \frac{P\,(T_R' - 32)(1\ BTU/lb\ WATER\text{-}°F)}{144\ BTU/lb\ ICE}$$

M_R = INCHES/INCH OF PRECIPITATION

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Project: W. I. [illegible]
Client Number: [illegible] Name: [illegible]
Date: [illegible]
Sheet ___ of ___
By: [illegible]

III WEATHER CONDITIONS PRIOR TO & DURING PMP

HMR 43 CHAPTER VIII GIVES PROCEDURES FOR ESTIMATING WIND, TEMPERATURE AND DEWPOINTS PRIOR TO AND DURING A PMF EVENT FOR SNOWMELT COMPUTATIONS. THESE CALCULATIONS FOLLOW

A. TEMPERATURE & DEWPOINT DURING PMP

(1) 51°F (12 HR 1000 mb Dewpt)

(2) 0.88" precipitable water (Wp)

(3) Percentage Ratios (FROM FIG 3-14)

6-HR PERIODS

1	2	3	4	5	6	7	8	9	10	11	12
1.04	1.00	.97	.95	.92	.90	.89	.87	.85	.84	.82	.81

(4) W(p) FOR TIME INCREMENTS

1	2	3	4	5	6	7	8	9	10	11	12
.91	.88	.85	.84	.81	.79	.78	.77	.75	.74	.72	.71

(5) MID-JANUARY 1000 mb TEMPERATURES (FIG 8-2)

1	2	3	4	5	6	7	8	9	10	11	12
51.7	51.0	50.3	50.2	49.5	49.1	48.8	48.6	48.1	47.9	47.3	47.0

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Project: M. C. [illegible]
Client Number [illegible] Name: Tailings Closure
Date: 7/18/9[illegible]
Sheet 4 of
By: [illegible]

(6) TEMPERATURE REDUCED TO 4000' AVERAGE BASIN ELEVATION

1	2	3	4	5	6	7	8	9	10	11	12
34.9	36.1	36.5	37.8	39.3	38.5	37.7	36.9	35.8	35.2	34.1	33.7

(7) REARRANGE FOR PMP SEQUENCE

BY TIME:	1	2	3	4	5	6	7	8	9	10	11	12
BY INTENSITY:	(7 & 8)		4	2	1	3	(5 & 6)		(9 & 10)		(11 & 12)	
RAINFALL	.5	.5	.9	1.7	3.5	1.1	.8	.7	.5	.4	.3	.2
TEMP	35.8	36.1	37.7	38.5	39.3	37.8	36.9	36.5	35.2	34.9	34.1	33.7

B. TEMPERATURES PRIOR TO PMP STORM

(1) 35.8°F

(2) JANUARY TEMPERATURE CHANGES PRIOR TO STORM (FROM FIG - 8-1)

NO CHANGES REQUIRED

(3) USE 35.8°F FOR 72 HR PRIOR TO STORM

C. DEWPOINTS PRIOR TO PMP STORM (FIG. 8-1)

(1) 6 HR - PERIOD PRIOR TO STORM

	1	2	3	4	5	6	7	8	9	10	11	12
ADJUST.	0	1	1	1	2	2	2	3	3	3	4	4
(2)	35.8	34.8	34.8	34.8	33.8	33.8	33.8	32.8	32.8	32.8	31.8	31.8

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Project: W. R. GRACE
Client Number: 02 Name: TAILINGS CLOSURE
Date: 7/17/92 Sheet C of
By: NTH

D. WINDS DURING PMP

1) BASIN AVE ELEVATION = 4000'
AVE PRESSURE = 870 mb

2) 6-hr JAN. ANEMOMETER WINDS @ 870 mb
= 35 Kts or 40 mph

3) JAN 6-hr % = 100%

4) 40 mph = MAXIMUM WIND VELOCITY

5) DURATION FACTORS FROM FIG 4-35
6-HR PERIOD

1	2	3	4	5	6	7	8	9	10	11	12
1.00	.92	.86	.79	.73	.68	.64	.61	.58	.56	.53	.52

6) CORRECTED WIND VELOCITIES

1	2	3	4	5	6	7	8	9	10	11	12
40	37	34	32	29	27	26	24	23	22	21	21

7) REARRANGED FOR PMP SEQUENCE

1	2	3	4	5	6	7	8	9	10	11	12
24	26	32	37	40	34	29	27	23	22	21	21

E. WINDS PRIOR TO PMP
USE 24 mph for 72 hours

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Project: [illegible]
Client Number 102 Name: TAILINGS CLOSURE
Date: 7/17/97
Sheet 6 of
By: TJH

IV PMF CALCULATIONS

DATA REGARDING RAINFALL AND WEATHER CONDITIONS WERE ENTERED INTO A SPREADSHEET TO CALCULATE RUN-OFF INTENSITIES. NO ALLOWANCE WAS MADE FOR INFILTRATION LOSSES.

CALCULATION OF THE PMF IS BASED ON THE UNIT HYDROGRAPH METHOD DESCRIBED IN "FLOOD HYDROLOGY MANUAL" U.S. BUREAU OF RECLAMATION, 1989. THIS IS THE SAME METHOD AS USED FOR THE THUNDERSTORM PMF BUT AMENDED TO INCLUDE SNOWMELT AND CALCULATED WITHOUT ALLOWANCE FOR INFILTRATION.

AN INITIAL EVALUATION OF PRE-STORM SNOW MELT SHOWED THAT IN THE ABSENCE OF INFILTRATION A SUBSTANTIAL DISCHARGE COULD BE PRODUCED (~ 190 CFS). SEE ATTACHED FIGURES. HOWEVER, THIS WILL BE READILY DISCHARGED BY THE BOX-CULVERT WITHOUT LOSS OF STORAGE CAPACITY.

THE HYDROGRAPH OF THE 3-DAY STORM PRODUCED A PEAK RUNOFF OF 3704 CFS. WHILE THE PEAK FLOW IS MUCH LOWER THAN THE THUNDERSTORM PMF, THE DURATION IS MUCH LONGER. WITHOUT AN EMERGENCY SPILLWAY THE RESERVOIR WILL SAFELY PASS A 0.53 RAIN ON SNOW PMF EVENT. THE PROPOSED EMERGENCY SPILLWAY PROVIDE ADDITIONAL FLOOD-ROUTING CAPACITY.

ATTACHMENT A

3-DAY GENERAL STORM
PMP WORKSHEETS

Table 6-1

Basin [illegible] Basin Size 94 Sq. Mi.

PART I. CONVERGENCE PMP (In.)

LECEMD

A Stinalation-elevation-barxler faecor. Figs. 3-37a to c.
B 24-hr. 1000-mb. 10-sq. mi. PMP. Figs. 3-23a through 3-31a.
C Product of A and B.
0 6/24-hr. ratios. Figs. 3-23b through 3-31b.
E 6-hr. 10-sq. mi. PMP. (Product of C and 0)
P Increnentsl percents. Table 3-3. 6-hr. periods through P_4, 12 hrs. for P_6, F_8, P_{10} and F_{12}
G Basin size reduction. Figs. 3-40a or 3-40b. First four periods only.
H Product of P and G (each of first four periods). No entry later periods.
I Accumulated increments. $I_2 = H_1 + H_2$, $I_3 = I_2 + H_3$, $I_4 = I_3 + H_4$, $I_6 = I_4 + F_6$, $I_8 = I_6 + F_8$, etc.
J Accumulated basin average convergence PMP, E x K_1, I_1, I_3, etc.

A = 0.62

	Through 1st period								Through 2d period				
	B	C	D	E	F_1	G_1	H_1	J_1	F_2	G_2	H_2	I_2	J_2
OCT	9.8	6.1	.60	3.7	1.00	1.00	1.00	3.7	.34	1.00	.34	1.34	5.0
NOV	8.8	5.5	.59	3.2	1.00	1.00	1.00	3.2	.35	1.00	.35	1.35	4.3
DEC	8.1	5.0	.58	2.9	1.00	1.00	1.00	2.9	.36	1.00	.36	1.36	3.9
JAN	7.8	4.8	.58	2.8	1.00	1.00	1.00	2.8	.36	1.00	.36	1.36	3.8
FEB	7.4	4.6	.58	2.7	1.00	1.00	1.00	2.7	.34	1.00	.36	1.36	3.7
MAR	7.6	4.7	.59	2.8	1.00	1.00	1.00	2.8	.35	1.00	.35	1.35	3.8
APR	9.0	5.6	.61	3.4	1.00	1.00	1.00	3.4	.33	1.00	.33	1.33	4.5
MAY	11.2	6.9	.625	4.3	1.00	1.00	1.00	4.3	.31	1.00	.31	1.31	5.6
JUNE	13.1	8.1	.665	5.4	1.no	1.00	1.00	5.4	.26	1.00	.26	1.26	6.8

	Through 3d period					Through 4ch period					Through 6th period		
	F_3	C_3	H_3	l_3	J_3	F_1	G_4	H_4	I_4	J_4	F_6	I_6	J_6
OCT	.20	1.00	.20	1.54	5.7	.14	1.00	.14	1.68	6.2	.19	1.87	6.9
NOV	.20	1.00	.20	1.55	5.0	.15	1.00	.15	1.70	5.4	.20	1.90	6.1
DEC	.21	1.00	.21	1.57	4.6	.15	1.00	.15	1.72	5.0	.21	1.93	5.6
JAN	.21	1.00	.21	1.57	4.4	.15	1.00	.15	1.72	4.8	.21	1.93	5.4
FEB	.21	1.00	.21	1.57	4.2	.15	1.00	.15	1.72	4.6	.21	1.93	5.2
MAR	.20	1.00	.20	1.55	4.3	.15	1.00	.15	1.70	4.8	.20	1.90	5.3
APR	.19	1.00	.19	1.52	5.2	.13	1.00	.13	1.65	5.6	.18	1.83	6.2
MAY	.18	1.00	.18	1.49	6.4	.12	1.00	.12	1.61	6.9	.16	1.77	7.6
JUNB	.14	1.00	.14	1.40	7.6	.10	1.00	.10	1.50	8.1	.13	1.63	8.8

	Through 8th period			Through 10th period			Through 12th period		
	F_a	I_8	J_8	F_{10}	I_{10}	J_{10}	F_{12}	I_{12}	J_{12}
OCT	.13	2.00	7.4	.10	2.10	7.8	.09	2.19	8.1
NOV	.14	2.04	6.5	.11	2.15	6.9	.09	2.24	7.2
DEC	.15	2.08	6.0	.12	2.20	6.4	.10	2.30	6.7
JAN	.15	2.08	5.8	.12	2.20	6.2	.10	2.30	6.4
FEB	.15	2.08	5.6	.12	2.20	5.9	.10	2.30	6.2
MAR	.14	2.04	5.7	.11	2.15	6.0	.09	2.24	6.3
APR	.12	1.95	6.6	.10	2.05	7.0	.08	2.13	7.2
MAY	.11	1.88	8.1	.10	1.98	8.5	.08	2.06	8.9
JUNE	.09	1.72	9.3	.08	1.80	9.7	.06	1.86	10.0

PART II. OROGRAPHIC PMP (In.)

LEGEND

K Basin slse factor. Figs. 4-39
L 6-hv. orographic Index. Grid average over basin. Figs. 4-33a to c.
M Product of K and L.

K = 1.00
L = .8
M = .8

Table 6-1 (Cont'd.)

Basin W.R. GRACE [illegible]

West of Cascade Divide

Table Q obtained by multiplying N times the accumulated orographic seasonal-durational factors below:

Accumulated factors

Per.	1	2	3	4	6	8	10	12
Hr.	6	12	18	24	36	48	60	72
OCT	1.08	2.04	2.91	3.67	4.96	5.97	6.76	7.37
NOV	1.07	2.02	2.88	3.64	4.91	5.92	6.70	7.30
DEC	1.04	1.97	2.80	3.54	4.77	5.75	6.51	7.09
JAN	1.00	1.89	2.69	3.40	4.59	5.53	6.26	6.82
FEB	1.00	1.89	2.69	3.40	4.59	5.53	6.26	6.82
MAR	.95	1.80	2.56	3.23	4.36	5.25	5.95	6.48
APR	.87	1.64	2.34	2.96	3.99	4.81	5.45	5.93
MAY	.76	1.44	2.04	2.58	3.49	4.20	4.76	5.18
JUNE	.68	1.29	1.83	2.31	3.12	3.76	4.26	4.64

East of Cascade Divide

	O	N	D	J	F	M	A	M	J
N	.98	.97	.94	.90	.90	.90	.95	1.00	[illegible]

Accumulated durational factors

Per.	1	2	3	4	6	8	10	12
Hr.	6	12	18	24	36	48	60	72
	1.00	1.89	2.69	3.40	4.59	5.48	6.09	6.46
P*	0.80	1.51	2.15	2.72	3.67	4.38	4.87	[illegible]

P* = M x durational factors

Multiply values in line P by percents in line N to get basin orographic PMP. Enter in table Q.

O. Accumulated Orographic PMP (in.)

Duration (hr.)

	6	12	18	24	36	48	60	72
OCT	.8	1.5	2.2	2.7	3.7	4.3	4.8	5.1
NOV	.8	1.5	2.1	2.6	3.6	4.2	4.7	5.0
DEC	.8	1.4	2.0	2.6	3.4	4.1	4.6	4.9
JAN	.7	1.4	1.9	2.4	3.3	3.9	4.4	4.7
FEB	.7	1.4	1.9	2.4	3.3	3.9	4.4	4.7
MAR	.7	1.4	1.9	2.4	3.3	3.9	4.4	4.7
APR	.8	1.4	2.0	2.6	3.5	4.2	4.6	4.9
MAY	.8	1.5	2.2	2.7	3.7	4.4	4.9	5.2
JUNE	.8	1.5	2.2	2.7	3.7	4.4	4.9	5.2

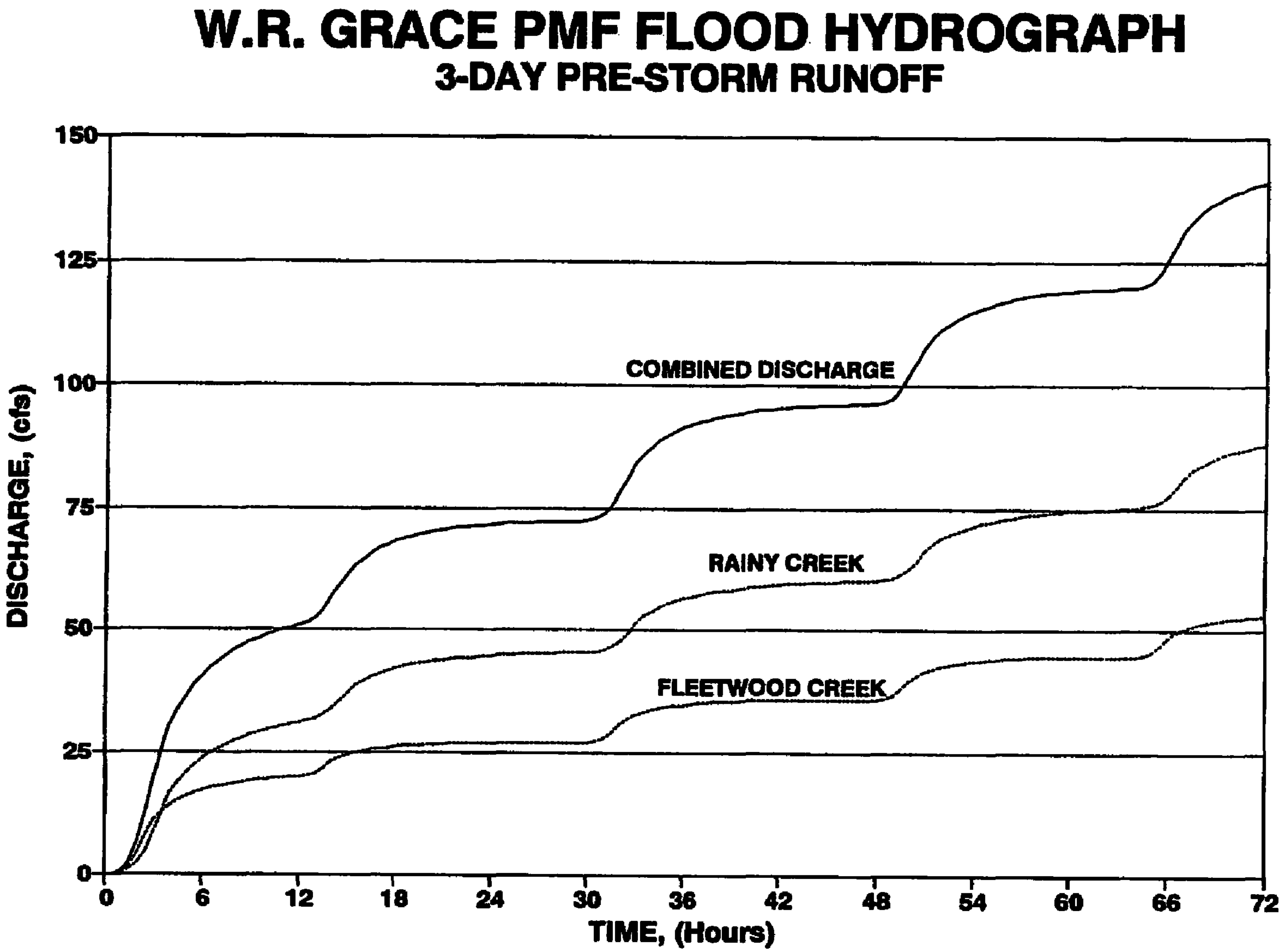
PART III. TOTAL PMP (in.)

Duration (hr.)

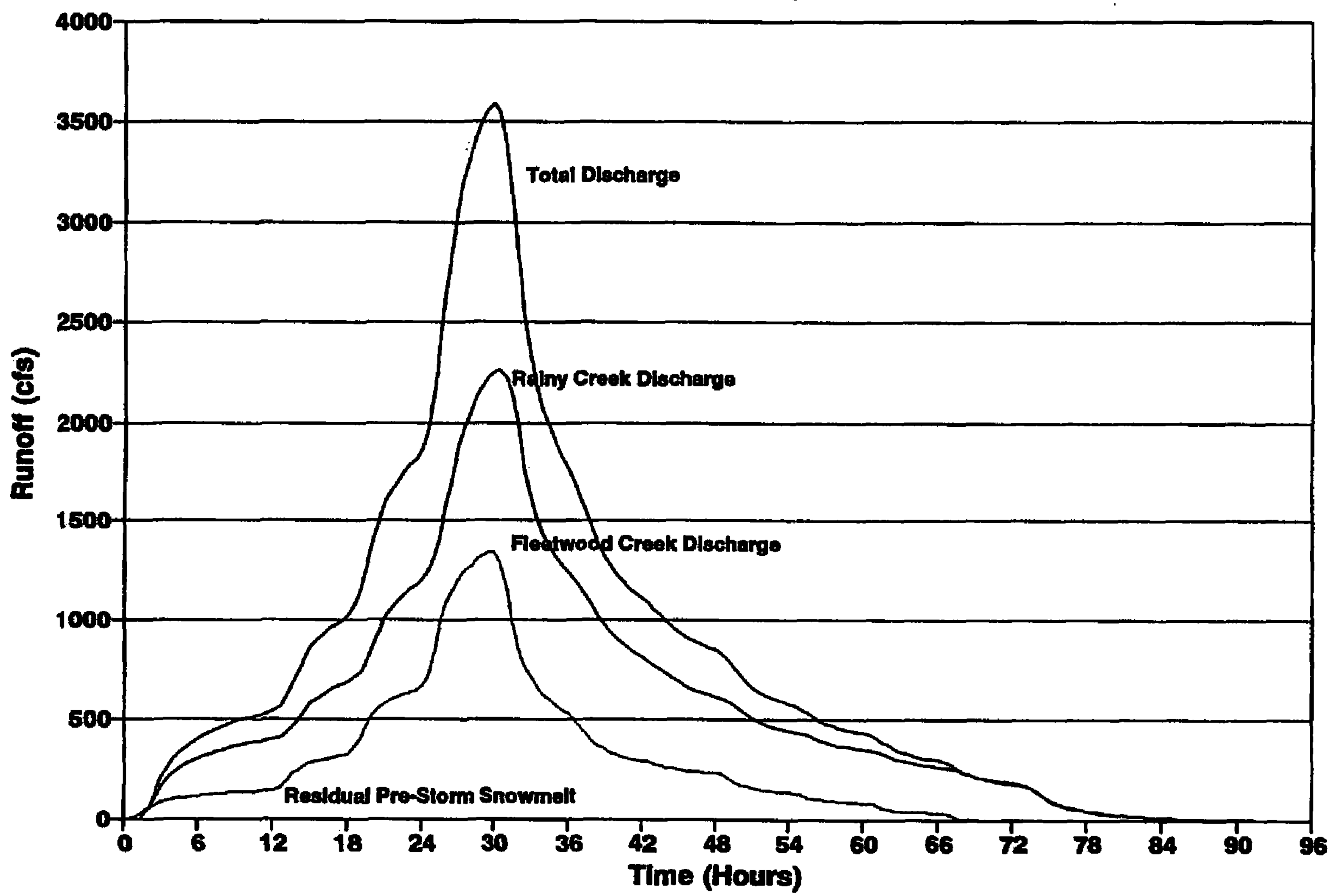
	6	12	18	24	36	48	60	72
OCT	4.5	6.5	7.9	8.9	10.6	11.7	12.6	13.7
NOV	4.0	5.8	7.1	8.0	9.7	10.7	11.6	12.2
DEC	3.7	5.3	6.1	7.6	9.0	10.1	11.0	11.6
JAN	3.5	5.2	6.3	7.2	8.7	9.7	10.6	11.1
FEB	3.4	5.1	6.1	7.0	8.5	9.5	10.3	10.9
MAR	3.5	5.2	6.2	7.2	8.6	9.6	10.4	11.0
APR	4.2	5.9	7.2	8.2	9.7	10.8	11.6	12.[illegible]
MAY	5.1	7.1	8.6	9.6	11.3	12.5	13.4	14.1
JUNE	6.2	8.3	9.8	11.8	12.5	13.7	14.6	15.[illegible]

ATTACHMENT B

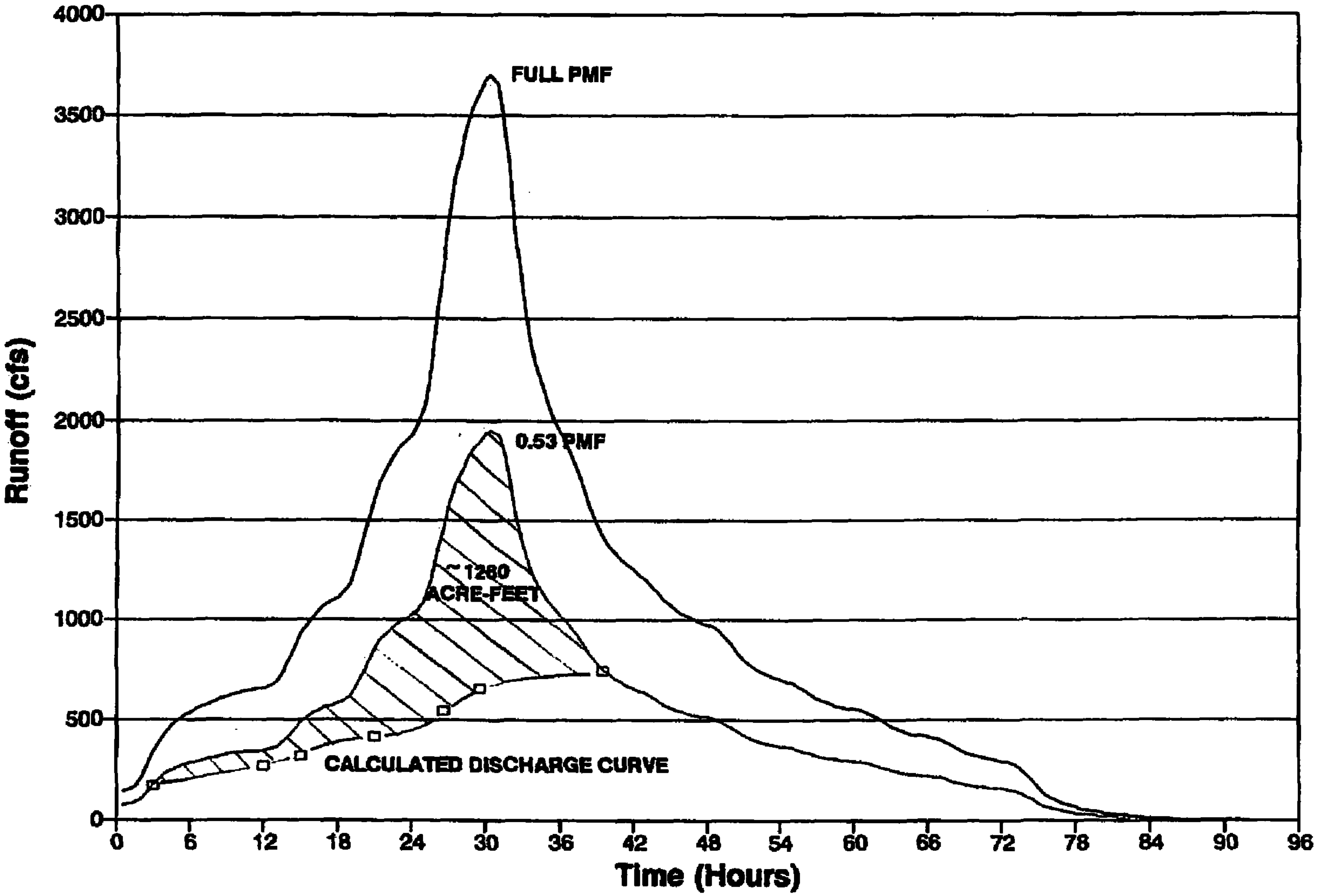
HYDROGRAPHS FOR RAIN ON SNOW PMP


W.R. GRACE PMF FLOOD HYDROGRAPH
3-DAY PRE-STORM RUNOFF
DISCHARGE, (cfs)
TIME, (Hours)
COMBINED DISCHARGE
RAINY CREEK
FLEETWOOD CREEK
0
25
50
75
100
125
150
0
6
12
18
24
30
36
42
48
54
60
66
72

W.R. GRACE HYDROGRAPH
RAIN ON SNOW PMF



W.R. GRACE HYDROGRAPH
MAXIMUM STORM CAPACITY



ATTACHMENT C

SPREADSHEET OUTPUT FOR RAIN ON SNOW PMP

Time Interval	Air Temp.	Dewpoint	Wind Speed	Increment Convective Snowmelt	Increment Rainfall	Increment Contact Snowmelt	Increment Intin.	Increment Runoff	Rainy Creek Unit Hydrograph	Fleetwood Creek Unit Hydrograph	Rainy Creek Discharge	Fleetwood Creek Discharge	Residual Pre-Storm Discharge	Total Discharge
(hours)	(F)	(F)	(mph)	(inches)	(inches)	(inches)	(inches)	(inches)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
0.00	35.8	35.8	24	0.0118	0.0417	0.0011		0.0546	0	0			141	
0.50	35.8	35.8	24	0.0119	0.0417	0.0011		0.0546	61	76	0	0	141	141
1.00	35.8	35.6	24	0.0118	0.0417	0.0011		0.0546	149	273	3	4	140	146
1.50	39.0	35.6	24	0.0110	0.0417	0.0011		0.0548	319	673	11	19	137	187
2.00	35.5	35.8	24	0.0110	0.0417	0.0011		0.0546	949	640	29	56	126	212
2.50	35.8	35.8	24	0.0119	0.0417	0.0011		0.0546	866	612	99	102	112	273
3.00	35.8	35.8	24	0.0110	0.0417	0.0011		0.0546	967	401	106	139	94	335
3.50	35.8	35.8	24	0.0119	0.0417	0.0011		0.0548	607	202	150	197	76	303
4.00	35.8	35.8	24	0.0110	0.0417	0.0011		0.0546	533	226	203	173	63	439
4.50	35.8	35.6	24	0.0110	0.0417	0.0011		0.0546	442	183	232	189	54	471
5.00	35.8	35.6	24	0.0119	0.0417	0.0011		0.0546	358	144	256	195	46	498
5.50	35.8	35.6	24	0.0110	0.0417	0.0011		0.0546	274	126	276	203	40	518
0.00	38.1	36.1	26	0.0137	00417	0.0012		0.0565	299	110	291	210	[illegible]5	538
5.50	38.1	56.1	26	0.0137	0.0417	0.0012		0.0565	217	04	305	217	31	993
7.00	36.1	36.1	26	0.0137	0.0417	0.0012		0.0565	194	61	317	222	27	567
7.50	36.1	36.1	26	0.0137	0.0417	0.0012		0.0565	171	70	328	226	24	060
8.00	36.1	36.1	26	0.0137	0.0417	0.0012		0.0565	153	57	339	233	21	593
8.50	38.1	36.1	26	0.0137	0.0417	0.0012		0.0565	128	53	349	236	19	605
9.00	36.1	36.1	26	0.0137	0.0417	0.0012		0.0505	127	49	358	241	16	615
0.50	36.1	36.1	28	0.0137	0.0417	0.0012		0.0565	114	39	368	244	14	625
10.00	36.1	36.1	26	0.0137	0.0417	0.0012		0.0565	99	33	373	247	13	633
10.80	36.1	36.1	26	0.0137	0.0417	0.0012		0.0565	99	29	380	249	11	639
11.00	36.1	36.1	26	0.0137	0.0417	0.0012		0.0585	85	21	389	251	10	646
11.90	30.1	36.1	26	0.[illegible]137	0.0417	0.0012		0.0585	77	16	391	292	8	691
12.00	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	66	11	395	253	7	656
12.30	37.7	37.7	32	0.0220	0.0750	0.0080		0.1006	64	7	402	296	6	566
13.00	37.7	37.7	32	0.0226	0.0700	0.0030		0.1006	59	0	412	270	5	686
13.50	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	54		430	300	5	734
14.00	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	46		457	337	4	798
14.50	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	45		498	384	3	866
13.00	37.7	37.7	32	0.0226	0.0750	0.0000		0.1006	40		544	362	3	928
15.50	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	34		582	395	2	979
16.00	37.7	37.7	32	0.[illegible]226	0.0750	0.0080		0.1006	34		607	409	2	1014
18.50	37.7	37.7	32	0.0226	0.0750	0.0030		0.1006	30		629	413	2	1043
17.00	37.7	37.7	32	0.0226	0.0750	0.0090		0.1006	26		046	419	1	1067
17.50	37.7	37.7	32	0.0326	0.0750	0.0090		0.1006	22		660	429	1	1088
10.00	38.5	36.5	37	0.0292	0.1417	0.0084		6.1772	17		873	430	1	1103
18.50	38.5	38.5	37	0.0292	0.1417	0.0084		0.1772	16		666	440	0	1126
10.00	36.5	36.9	37	0.0292	0.1417	0.0084		0.1772	11		708	464	0	1173

Time Interval (hours)	Air Temp. (F)	Dewpoint (F)	Wind Speed (mph)	Increment Convective Snowmelt (inches)	Increment Rainfall (inches)	Increment Contact Snowmelt (inches)	Increment Infilt. (inches)	Increment Runoff (inches)	Rainy Creek Unit Hydrograph (cfs)	Fleetwood Creek Unit Hydrograph (cfs)	Rainy Creek Discharge (cfs)	Fleetwood Creek Discharge (cfs)	Residual Pre-Storm Discharge (cfs)	Total Discharge (cfs)
19.90	38.5	38.8	37	0.8282	0.1417	0.0084		0.1772	7		741	519	0	1280
20.00	38.8	38.8	37	0.8292	0.1417	0.0084		0.1772	3		790	588	0	1378
20.50	36.5	38.8	37	0.0202	0.1417	0.0084		0.1772	0		863	835	0	1498
21.00	36.5	38.8	37	0.0202	0.1417	0.0084		0.1772			043	888	0	1811
21.50	38.6	38.5	37	0.8292	0.1417	0.0084		0.1T72			1010	892	0	1702
22.00	38.8	38.9	37	0.0992	0.1417	0.0084		0.1772			1055	711	0	1788
22.80	38.8	38.9	37	0.8292	0.1417	0.0084		0.1772			1093	728	0	1819
23.00	38.5	30.5	37	0.0292	0.1417	0.0084		0.1772			1124	738	0	1882
23.50	38.5	38.8	37	0.0292	0.1417	0.0084		0.1772			1149	749	0	1897
24.00	39.3	38.8	40	0.0351	0.2017	0.0148		0.3415			1172	798	0	1920
24.80	39.3	39.8	40	0.0351	0.2917	0.0148		0.8419			1201	778	0	1979
29.80	39.8	39.8	40	0.0351	0.2017	0.0148		0.8415			1242	829	0	2071
25.50	39.3	39.8	40	0.0351	0.2917	0.0148		0.8415			1310	945	0	2259
20.00	39.8	39.3	40	0.0351	05917	0.0148		0.3415			1414	1087	0	2502
26.50	39.8	30.8	40	0.0351	0.2917	0.0148		0.3418			1589	1192	0	2781
27.00	38.8	39.8	40	0.0351	05917	0.0148		0.8415			1739	1281	0	3000
27.80	39.3	39.8	40	0.0351	0.8917	0.0148		0.3419			1882	1312	0	3194
28.00	39.3	39.8	40	0.0391	0.2917	0.0148		0.8415			1979	1352	0	3330
28.80	39.8	39.8	40	0.0351	0.8917	0.0148		0.8415			2080	1384	0	3444
29.80	39.8	39.8	40	0.0351	02017	0.0148		0.8415			2128	1409	0	3538
29.90	39.8	39.8	40	0.0351	0.2917	0.0148		0.8419			2178	1432	0	3810
30.00	37.8	37.8	34	0.0242	0.0917	0.0037		0.1188			2228	1491	0	3877
30.50	37.8	37.8	34	0.0242	0.0917	0.0037		0.1108			2254	1449	0	3704
31.00	37.8	37.8	34	0.0242	0.0817	0.0037		0.1108			8250	1402	0	3881
31.80	37.8	37.8	34	0.0242	0.0917	0.0037		0.1198			2221	1284	0	3485
32.00	37.8	37.8	34	0.8242	0.0917	0.0037		0.1188			2128	1087	0	3215
32.80	37.8	37.8	34	0.0242	0.0017	0.0097		0.1198			1859	960	0	2820
33.00	37.8	37.8	34	0.0242	0.0917	0.0037		0.1198			1789	078	0	2847
33.80	37.8	37.8	34	0.0242	0.0917	0.0037		0.1198			1811	820	0	2431
34.00	37.8	37.8	34	0.0242	0.0917	0.0037		0.1198			1512	779	0	2287
34.50	37.8	37.8	34	0.8242	0.0917	0.0087		0.1198			1431	739	0	2171
35.00	37.8	37.8	34	0.0242	00917	0.0037		0.1198			1388	711	0	2079
35.50	37.8	37.8	34	0.0242	00917	0.0037		0.1198			1321	888	0	2007
80.00	30.0	38 5	29	0.0179	0.0887	0.0023		0.0888			1278	883	0	1839
36.50	38.9	36.9	29	0.0170	0.0887	0.0023		0.0888			1238	841	0	1878
37.00	38.8	38.8	29	0.0170	0.0887	0.0023		0.0888			1200	814	0	1814
37.80	38.8	38.8	29	0.0179	0.0887	0.0023		0.0888			1181	978	0	1737
30.00	38.9	38.8	29	0.0179	0.0887	0.0023		0.0888			1117	538	0	1853
38.50	38.8	38.8	20	0.0179	0.0887	0.0023		0.0888			1088	504	0	1972

Time Interval	Air Temp.	Dewpoint	Wind Speed	Increment Convective Snowmelt	Increment Rainfall	Increment Contact Snowmelt	Increment Infilt.	Increment Runoff	Rainy Creek Unit Hydrograph	Fleetwood Creek Unit Hydrograph	Rainy Creek Discharge	Fleetwood Creek Discharge	Residual Pre-Storm Discharge	Total Discharge
(hours)	(F)	(F)	(mph)	(inches)	(inches)	(inches)	(inches)	(inches)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
39.00	36.9	36.8	29	0.0179	0.0667	0.0023		0.0866			1015	461	0	1496
39.90	36.9	36.9	29	0.0179	0.0667	6.0023		0.0868			969	463	0	1431
40.00	36.9	36.9	29	0.0179	0.0607	0.0023		0.0866			935	448	0	1303
40.90	36.9	36.9	29	0.0179	0.0667	0.0023		0.0868			904	438	0	1340
41.00	36.9	36.9	29	0.0170	0.0667	0.0023		0.0608			676	426	0	1304
41.90	36.9	36.0	29	0.0170	0.0667	0.0023		0.0806			855	418	0	1273
42.00	36.9	36.5	27	0.0159	0.0583	0.0018		0.0756			635	412	0	1247
42.50	36.5	36.9	27	0.0155	0.0583	0.0018		0.0756			819	407	0	1222
43.00	86.5	36.5	27	0.0155	0.0983	0.0018		0.0796			796	401	0	1197
43.90	36.9	36.5	27	0.0159	0.0583	0.0018		0.0756			776	391	0	1167
44.00	36.5	36.5	27	0.0159	0.0563	0.0018		0.0756			755	380	0	1139
44.90	36.9	36.9	27	0.0159	6.0583	0.0018		0.0756			731	371	0	1102
49.00	36.9	38.9	27	0.0155	0.0583	0.0018		0.0756			707	365	0	1072
49.50	36.5	50.5	27	0.0155	0.0583	0.0018		0.0796			667	361	0	1046
46.00	36.9	36.9	27	0.0155	0.0563	0.0016		0.0796			670	357	0	1027
46.90	36.5	36.9	27	0.0195	0.0583	6.0016		0.0796			855	354	0	1009
47.00	36.5	36.5	27	0.9159	0.0565	0.0018		0.0756			643	392	0	995
47.50	36.9	36.5	27	0.0159	0.0663	0.0018		0.0756			632	350	0	982
48.00	35.9	39.2	23	0.0097	0.0417	0.0009		0.0923			624	346	0	972
48.80	35.2	35.2	23	0.0097	0.9417	0.0005		0.0923			614	345	0	959
40.00	35.9	35.9	23	0.0007	0.0417	0.0009		0.0323			604	338	0	942
40.50	35.9	35.2	23	0.0007	0.0417	0.0009		0.0529			592	321	0	913
56.00	35.9	35.9	23	0.0007	0.0417	0.0009		0.0523			575	301	0	876
90.50	39.2	35.9	23	0.0007	0.0417	0.0005		0.0523			552	286	0	638
51.00	35.2	35.2	23	0.0057	0.0417	0.0009		0.0323			926	276	0	603
91.90	35.2	39.9	23	0.0097	0.0417	0.0009		0.0523			505	269	0	774
92.00	39.2	35.9	23	0.0057	0.0417	0.0005		0.0525			480	263	0	794
92.90	39.2	35.2	23	0.0007	0.0417	0.0009		0.0923			476	259	0	737
93.00	35.9	35.9	23	0.0097	0.0417	0.0009		0.0523			468	255	0	723
93.90	35.9	35.2	23	0.0097	0.0417	0.0000		0.0523			460	252	0	712
54.00	34.9	34.9	22	0.0085	0.0333	0.0007		0.0425			492	240		702
54.50	34.9	34.9	22	0.0085	0.0333	0.0007		0.0425			446	246		692
55.00	34.9	34.9	22	0.0085	0.0333	0.0007		0.0425			430	242		680
55.90	34.9	34.9	22	0.0085	0.0333	0.0007		0.0429			431	233		864
68.00	34.9	34.9	22	0.9065	0.0333	0.0007		0.0425			421	224		643
56.90	34.9	34.9	22	0.0085	0.0333	0.0007		0.0429			409	217		628
57.00	34.9	34.9	22	0.0085	0.0333	0.0007		0.0429			386	212		808
57.50	34.9	34.9	22	0.0085	0.0333	0.0007		0.0429			365	208		593
58.00	34.9	34.9	22	0.9065	0.0333	0.0007		0.0425			377	205		582

Time Interval	Air Temp.	Dewpoint	Wind Speed	Increment Convective Snowmelt	Increment Rainfall	Increment Contact Snowmelt	Increment Infilt.	Increment Runoff	Rainy Creek Unit Hydrograph	Fleetwood Creek Unit Hydrograph	Rainy Creek Discharge	Fleetwood Creek Discharge	Residual Pre-Storm Discharge	Total Discharge
(hours)	(F)	(F)	(mph)	(inches)	(inches)	(inches)	(inches)	(inches)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
08.50	34.5	34.9	22	0.0085	0.0333	0.0007		0.0425			370	202		973
80.00	34.5	34.5	22	0.0085	0.0333	0.0007		0.0425			365	200		969
99.50	34.5	34.5	22	0.0089	0.0333	0.0007		0.0425			380	109		999
60.00	34.1	34.1	21	0.0099	0.0250	0.0004		0.0313			356	187		953
60.90	34.1	34.1	21	0.0059	0.0250	0.0004		0.0313			351	105		547
01.00	84.1	34.1	21	0.0050	0.0250	0.0004		0.0313			346	192		538
01.90	34.1	34.1	21	0.0090	0.0250	0.0004		0.0313			340	163		523
62.00	34.1	34.1	21	0.0059	0.0250	0.0004		0.0313			331	173		904
62.50	34.1	34.1	21	0.0058	0.0250	0.0004		0.0313			319	166		465
63.00	34.1	34.1	21	0.0099	0.0290	0.0004		0.0313			306	161		467
63.50	34.1	34.1	21	0.0099	0.0200	0.0004		0.0313			295	156		452
64.00	34.1	34.1	21	0.0099	0.0250	0.0004		0.0313			287	155		442
64.50	34.1	34.1	21	0.0099	0.0250	0.0004		0.0313			280	152		433
63.00	34.1	34.1	21	0.0099	0.0250	0.0004		0.0313			275	151		426
69.50	34.1	34.1	21	0.0059	0.0250	0.0004		0.0313			271	149		420
68.00	33.7	33.7	21	0.0048	0.0167	0.0002		0.0216			267	148		414
68.50	33.7	33.7	21	0.0048	0.0167	0.0002		0.0210			263	146		406
67.00	33.7	33.7	21	0.0046	0.0167	0.0002		0.0216			256	142		401
67.50	33.7	33.7	21	0.0046	0.5167	0.0002		0.0216			253	135		386
60.00	33.7	33.7	21	0.0048	0.0167	0.0002		0.0216			245	128		371
68.50	99.7	33.7	21	0.0048	0.0167	0.0002		0.0216			235	120		355
69.00	33.7	33.7	21	0.0048	0.0167	0.0002		0.0216			224	119		339
69.50	33.7	33.7	21	0.0048	0.0167	0.0002		0.0216			214	112		326
70.00	33.7	33.7	21	0.0048	0.0167	0.0002		0.0216			208	110		317
70.50	33.7	33.7	21	0.0040	0.0167	0.0002		0.0216			202	107		310
71.00	33.7	33.7	21	0.0046	0.0167	0.0002		0.0216			196	106		303
71.50	33.7	33.7	21	0.0048	0.0167	0.0002		0.0218			194	104		298
72.00											190	103		294
72.50											166	101		287
73.00											161	95		276
73.50											173	62		239
74.00											160	64		224
74.50											141	49		100
70.00											119	39		190
75.50											99	32		131
70.00											85	28		111
76.50											74	22		05
77.00											64	16		63
77.50											57	19		72

Time Interval	Air Temp.	Dewpoint	Wind Speed	Increment Convective Snowmelt	Increment Rainfall	Increment Contact Snowmelt	Increment Infilt.	Increment Runoff	Rainy Creek Unit Hydrograph	Fleetwood Creek Unit Hydrograph	Rainy Creek Discharge	Fleetwood Creek Discharge	Residual Pre-Storm Discharge	Total Discharge
(hours)	(F)	(F)	(mph)	(inches)	(inches)	(inches)	(inches)	(inches)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
78.00											81	13		83
78.50											45	10		55
79.00											40	9		49
79.50											39	7		43
80.00											32	8		37
80.50											28	5		33
81.00											29	4		29
81.50											22	3		25
82.00											20	2		22
82.50											17	1		19
83.00											15	1		18
83.50											13	0		14
84.00											12	0		12
84.90											10	0		10
88.00											9	0		9
89.50											7	0		7
80.00											8	0		8
88.50											5	0		9
87.00											4	0		4
87.50											4	0		4
88.00											3	0		3
88.90											2	0		2
89.00											2	0		2
89.50											1	0		1
90.00											1	0		1
90.50											1	0		1
91.00											0	0		0
01.50											0	0		0
02.00											0	0		0
Totals:				2.4	11.1	0.4	0.0	13.0						

A3-Reference 3

ENGINEERING ANALYSIS OF FLOOD ROUTING ALTERNATIVES

for the

W.R. GRACE VERMICULITE TAILINGS IMPOUNDMENT LIBBY, MONTANA

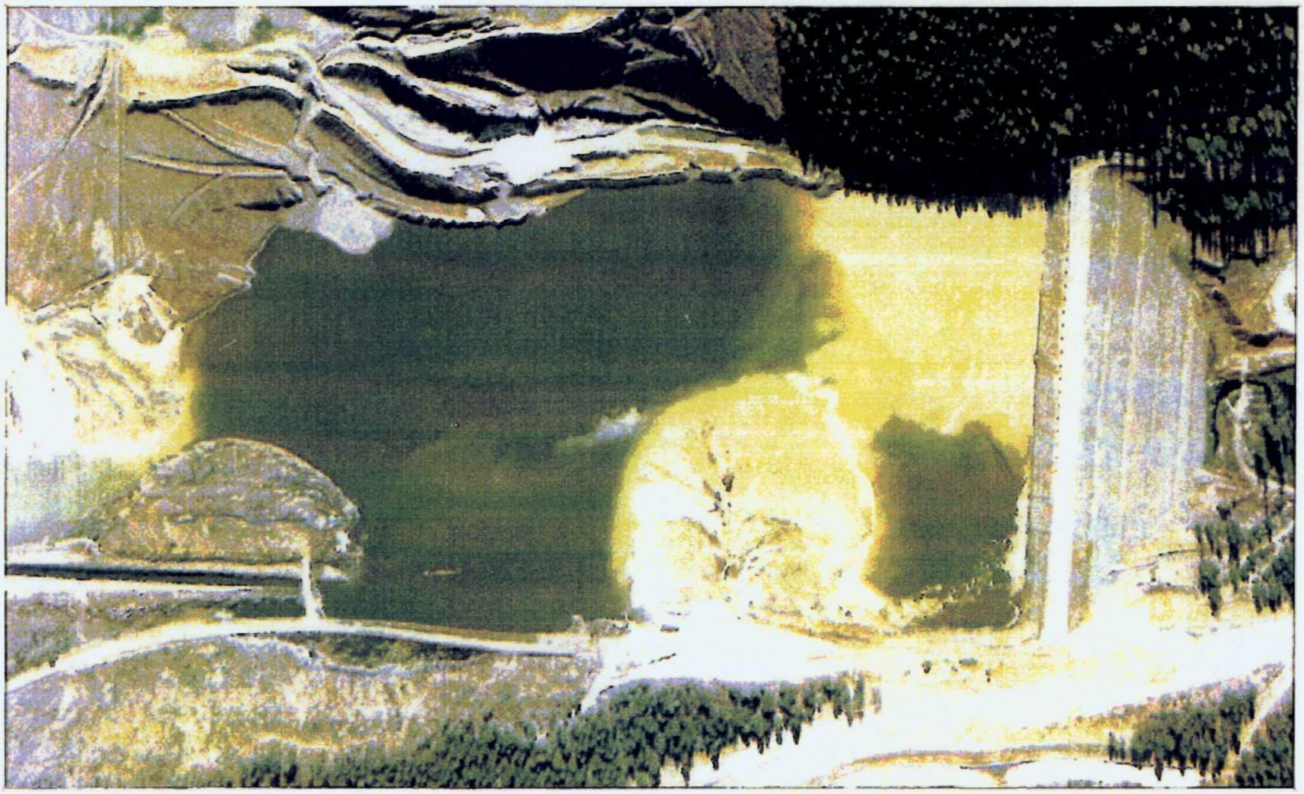


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Table of Contents

Section	Description	Page

1.0 EXECUTIVE SUMMARY 1-1

2.0 INTRODUCTION 2-1
- 2.1 OVERVIEW/PROJECT OBJECTIVES 2-1
- 2.2 PROJECT LOCATION AND DESCRIPTION 2-2
- 2.3 SITE HISTORY/BACKGROUND 2-5

3.0 HYDROLOGIC EVALUATION 3-1
- 3.1 HYDROLOGIC PARAMETERS 3-1
- 3.2 DESIGN STORMS 3-3
  - 3.2.1 10-Year Event 3-4
  - 3.2.2 100-Year Event 3-4
  - 3.2.3 Probable Maximum Flood 3-7
- 3.3 TAIUNGS IMPOUNDMENT CAPACITY 3-9
- 3.4 DAM SAFETY REQUIREMENTS 3-10
- 3.5 PROPOSED DESIGN FLOWS 3-12

4.0 FLOOD ROUTING 4-1
- 4.1 OVERVIEW OF ALTERNATIVES 4-1
- 4.2 FULL DIVERSION ALTERNATIVES 4-3
  - 4.2.1 Description of Design Concepts 4-3
  - 4.2.2 Evaluation of the Full Diversion Alternatives 4-5
- 4.3 CHANNEL RECONSTRUCTION THROUGH THE IMPOUNDMENT . . . 4-11
  - 4.3.1 Description of Design Concepts 4-11
  - 4.3.2 Evaluation of Alternatives for Channel Reconstruction In the Tailings 4-14
- 4.4 PARTIAL DIVERSION 4-18
  - 4.4.1 Description of Conceptual Designs 4-18
  - 4.4.2 Evaluation of Partial Diversion Alternatives 4-20
- 4.5 SUMMARY/CONCLUSIONS 4-22

5.0 PROJECT DESIGN - PREFERRED ALTERNATIVE 5-1
- 5.1 GENERAL DESIGN APPROACH 5-1
- 5.2 TAIUNGS IMPOUNDMENT 5-2

5.3 INLET CHANNEL 5-2
5.4 CONTROL STRUCTURE 5-3
5.4.1 100-Year Event 5-3
5.4.2 Probable Maximum Flood 5-4
5.5 OUTLET CHANNEL 5-6
5.6 EMERGENCY SPILLWAY 5-6
5.7 REVEGETATION 5-7
5.8 STABIUZATION/EROSION CONTROL 5-8
5.9 OTHER CLOSURE ACTIVITIES 5-9

6.0 POST-CLOSURE CARE 6-1
6.1 POST-CLOSURE MANAGEMENT 6-1
6.2 WATER QUALITY MONITORING PROGRAM 6-1
6.3 MAINTENANCE 6-1

7.0 REFERENCES 7-1

List of Tables

Number	Description	Page
Table 3.1.	Hydrologic parameters for Rainy Creek and Fleetwood Creek drainage areas impounded by the tailings dam.	3-3
Table 3.2.	Surface water runoff for a 10-year, 24-hour precipitation event using SCS Type II rainfall distribution.	3-5
Table 3.3.	Surface water runoff for a 100-year, 24-hour precipitation event using SCS Type II rainfall distribution.	3-6
Table 3.4.	Surface water runoff for a 6-hour PMF event (10.7 in.) using the storm distribution hyetograph of Figure 3.4.	3-8
Table 3.5.	Storage capacity of the tailings impoundment/reservoir.drainage areas impounded by the tailings dam.	3-10
Table 3.6.	Emergency spillway inflow design flood(s) from Table A of the Montana Dam Safety regulations, Rule 36.14.502.	3-11
Table 3.7.	Design flood volumes proposed for flood routing altematives analysis and conceptual design.	3-13
Table 4.1.	Summary of alternatives considered for flood routing.	4-2
Table 4.2.	Advantages associated with a full diversion flood routing system.	4-8
Table 4.3.	Disadvantages associated with full diversion flood routing system	4-9
Table 4.4.	Advantages associated with routing floods through the tailings impoundment.	4-16
Table 4.5.	Disadvantages associated with routing floods through the tailings impoundment.	4-17
Table 4.6.	Advantages associated with partial diversion flood routing systems.	4-21
Table 4.7.	Disadvantages associated with partial diversion flood routing systems.	4-21
Table 5.1.	Flood routing parameters for various routing alternatives.	5-4

List of Figures

Number	Description	Page
Figure 2.1	Location of the W.R. Grace Project Area	2-3
Figure 2.2	W.R. Grace tailings impoundment. USGS Vermiculite Mountain, Mont Quadrangle, Lincoln Co.	2-4
Figure 3.1	Drainage areas for Rainy Creek and Fleetwood Creek.	3-2
Figure 3.2	Surface water runoff hydrographhs and rainfall intensity for a 10-year, 24-hour storm (2.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.	3-5
Figure 3.3	Surface water runoff hydrographs and rainfall intensity for a 100-year, 24-hour storm (2.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.	3-6
Figure 3.4	Storm hyetograph for a 6-hour PMF event (10.7 in,) in the Rainy Creek and Fleetwood Creek drainage basins.	3-8
Figure 3.5	Surface water mnoff hydrographs for a 6-hour PMF event (10.7 in.) in the Rainy Creek and Fleetwood Creek watersheds.	3-9

List of Drawing Plates

Number	Description
Plate 1.	Plan view of project area delineated by base grid system.
Plate 2.	Plan view of conceptual full diversion flood routing system employing a single diversion dam below the confluence of Rainy Creek and Fleetwood Creek.
Plate 3.	Cross-section A-A' showing typical section of diversion dam for a full diversion flood routing system.
Plate 4-A.	Plan view of full diversion dam and channel to deliver Fleetwood Creek to the Rainy Creek diversion dam.
Plate 4-B.	Plan view of Rainy Creek diversion system employing a dam upstream of tailings.
Plate 5.	Typical cross-section of west diversion channel showing limits of excavation.
Plate 6.	Typical section of the east side full diversion channel constructed partially within the fine tailings impoundment.
Plate 7.	Typical section of the east side full diversion channel constructed in bedrock, outside of the fine tailings.
Plate 8.	Plan view of flood routing through the impoundment in a reconstructed channel (Alternate lla), showing location of inflow and outflow channels, and control structure.
Plate 9.	Plan view of flood routing system through the impoundment with low profile dike(Altemate lib) showing the dike and revised inflow channel.
Plate 10.	Plan view of the outlet/controi structure over the dam face for routing floods through the impoundment (Altemate lie).
Plate 11.	Typical section for a partial diversion channel at west abutment area.
Plate 12.	Section showing centerline of channel for the proposed flood routing system for the vermiculite tailings impoundment.
Plate 13.	Typical cross-section of the inflow channel for the proposed flood routing system.
Plate 14.	Typical cross-section of the discharge control structure for the proposed flood routing system.
Plate 15.	Typical cross-section of the outflow channel for the flood routing system.
Plate 16.	Typical cross-section of the emergency relief spillway for the vennlculite tailings impoundment.

Appendices

Number	Description
Appendix A	Hydrologic Modeling Results
Appendix B	Probable Maximum Flood Calculations
Appendix C	Control Structure and Emergency Spillway Calculations
Appendix D	Flood Routing Results
Appendix E	Standard Drawing-SCS Drop Structure

1.0 EXECUTIVE SUMMARY

An engineering analysis of flood routing altematives was completed at the W.R. Grace vermiculite tailings impoundment, near Libby, Montana, to investigate the various altematives for routing floods through the tailings impoundment following closure. W.R. Grace has ceased mining and milling operations at the site and wishes to complete closure operations and requirements during 1992 in order to obtain bond release.

Regolatory agencies, including the Department of State Lands (DSL), USDA Forest Service, and others have raised concerns over the mine closure, particularly the closure of the tailings impoundment. These concerns include:

- asbestiform fiber contamination in surface water from the coarse tailings dump and fine tailings impoundment;
- long-term stability and integrity of the dam, primarily with regard to saturation and seepage failure;
- iucreased sedimentation of downstream areas from the impoundment;
- safety; and finally,
- setting a precedence for other tailings impoundments.

In order to address these issues, an engineering analysis of flood routing altematives was conducted. The purpose of the engineering analysis was to objectively examine the various altematives for routing Rainy Creek and Fleetwood Creek flows through the area affected by the vermiculite tailings impoundment, and to present a conceptual plan of the preferred alternative. The analysis addressed the issues of hydrology and flood routing, dam safety, short-term and long-term environmental impact, construction feasibility, costs, long-term stability and erosion control, and proposed reclamation methods and practices.

The impoundment is situated on Rainy Creek, immediately below the confluence with Fleetwood Creek, and impounds approximately 9.4 square miles of the Rainy Creek drainage area. A design flood of 0.5 PMF, calculated at 5838 cfs, was selected as the inflow volume that would be used for flood routing through the impoundment.

The investigation determined that the best method to safely pass a design storm of this magnitude in a stable manner, while assuring the long-term integrity of the dam, is to route the storm <u>through</u> the impoundment using controlled outflow stmctnres. By using the impoundment to temporarily store peak inflows, outflow volumes can be reduced to a fraction of the 0.5 PMF peak inflow volume.

Routing the floods through the impoundment using controlled outflow stmctures provided the safest and most cost effective method of flood routing for the tailings impoundment while addressing the majority of the regulatory concerns. Significant advantages include:

- Provides a higher level of public safety than other altematives while assuring the long-term integrity of the tailings dam and retaining a relatively straightforward design;
- Provides a cost-effective, relatively straightforward method of safely handling storm flows;
- During a 0.5 PMF event this design is geotechnically the most stable of the altematives;
- System is capable of handling floods larger than the design flood of 0.5 PMF with the addition of an emergency spillway;
- Outflows are considerably less than 0.5 PMF due to flood routing, allowing for a smaller, more cost effective channel, and less downstream disturbance during major events;
- Environmental disturbance is kept to a minimum with the a smaller, more natural outflow channel;
- The remaining impoundment wetland promotes surface water improvement through natural filtration and settiement;
- Least overall maintenance of the altematives;
- Minimal water loss to infiltration; and,
- Impoundment wetland would provide excellent wildlife habitat.

2.0 INTRODUCTION

2.1 OVERVIEW/PROJECT OBJECTIVES

W.R. Grace and Company, Zonolite Division, Libby, Montana, has retained Schafer and Associates, Bozeman, Montana, to perform an Engineering Analysis of Flood Routing Alternatives for Rainy Creek and Fleetwood Creek, which have been affected by a vermiculite tailings impoundment. The impoundment was constructed to provide process water and settle tailings at W.R. Grace's vermiculite mining/milling operations northeast of Libby. Currently, Rainy Creek is intercepted above the impoundment, and diverted around the tailings impoundment through a culvert constructed of 48 and 52 inch diameter corrugated metal pipe, re-entering the original channel below the tailings dam. Fleetwood Creek enters the impoundment through a constructed diversion channel.

W.R. Grace has ceased operations at the entire mining, milling, and shipping facilities, and has begun implementing reclamation and closure measures at the site. It is the desire of W.R. Grace to complete all reclamation and closure requirements during 1992, and obtain bond release for the entire project area and facilities, including the tailings impoundment.

Regulatory agencies, including the Department of State Lands (DSL), USDA Forest Service, and others have raised concerns over the mine closure, particularly the closure of the tailings impoundment. These concerns include:

- asbestiform fiber contamination in surface water from the coarse tailings dump and fine tailings impoundment;
- long-term stability and integrity of the dam, primarily with regards to saturation and seepage failure;
- increased sedimentation of downstream areas from the impoundment;
- safety; and,
- setting a precedence for other tailings impoundments.

In order to address these issues, an engineering analysis of flood routing alternatives was conducted. The objectives of the engineering analysis are to examine the various alternatives for routing Rainy Creek and Fleetwood Creek flawa through the area affected by the vermiculite tailings impoundment, and to present a conceptual plan of the preferred alternative. The analysis will address the issues of hydrology and flood routing, dam safety, environmental disturbance, construction feasibility, costs, long-term stability, erosion control, and proposed reclamation methods and practices. (Note: the issues of water quality and tailings dam stability are addressed in separate investigations titied "W.R. Grace Vermiculite Mine Closure Water Quality Monitoring Plan" (Hudson, 1991) and "Geotechnical Evaluation, W.R. Grace Dam, Rainy Creek, Montana" (Vahdani, 1992) respectively.

Various alternatives for collecting and routing Rainy and Fleetwood Creeks aroumd or through the impoundment will be reviewed, with advantages and disadvantages considered and discussed. The ultimate objective is to provide a method of passing storm flows through the impoundment area assuring the integrity of the dam without producing significant environmental impacts in the form of water quality degradation or distmrbances to local terrain.

- Our approach to meeting this objective is as follows:
- First, select suitable storm events which will be used as design criteria, determine size, and calculate runoff volumes for these storms (Chapter 3),
- Second, define and compare conceptual approaches and select a preferred alternative for detailed description (Chapter 4),
- Third, define essential elements of design for the preferred alternative and discuss possible altematives for implementing details of design (Chapter 5),
- Finally, propose maintenance procedures which will be implemented to provide for the perpetual safety of the implemented closure plan (Chapter 6),

2.2 PROJECT LOCATION AND DESCRIPTION

The vermiculite tailings impoundment is part of W.R. Grace's Construction Products Division vermiculite operations. The tailings impoundment encompasses approximately 70 acres within the drainage basin(s) of Rainy and Fleetwood Creeks. The site is located approximately seven miles east northeast of Libby, Montana, within the SW 1/4 of Section 15, and tiie NW 1/4 of Section 22, Township 31 Nortii, Range 30 West, Lincohi County, Montana. The site is accessed by State Highway 37, and USPS Road No. 401. The impoundment lies entirely within patented mine property owned by W.R. Grace and Company. Surrounding public land is managed by the USDA Forest Service, Libby Ranger District. See Figures 2.1 and 2.2.

The tailings impoundment is located immediately below the confluence of Rainy Creek and Fleetwood Creek. After leaving the mine property, Rainy Creek flows toward the southwest and enters the Kootenai River about 2 1/2 miles downstream of the dam, and about 5 1/2 miles upstream of Libby. The Kootenai River is a tributary of the Clark Fork

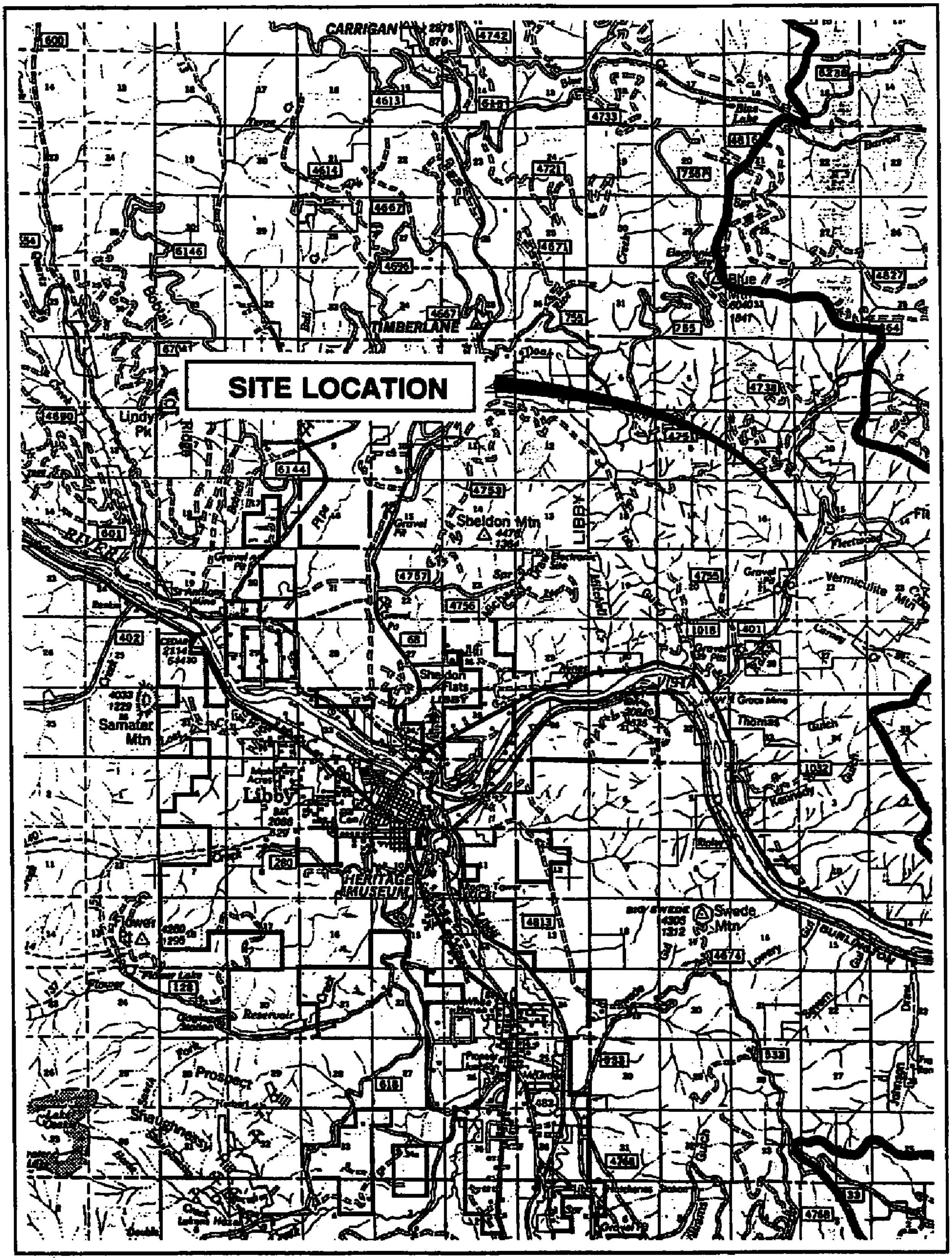


Figure 2.1 Location of the W.R. Grace Project Area.

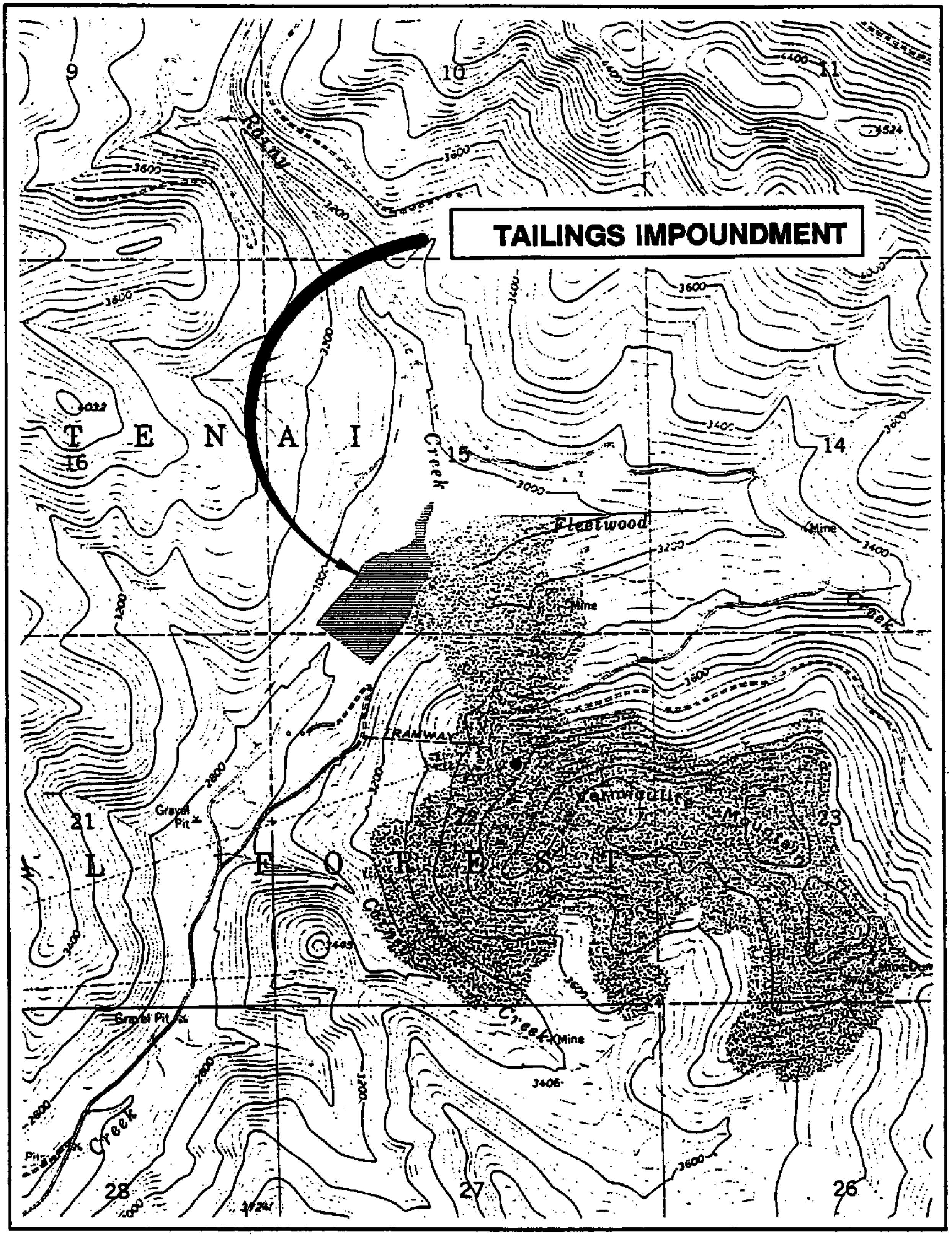


Figure 2.2 **W. R. Grace Vermiculite Tailings Impoundment. USGS Vermiculite Mountain, Mont Quadrangle, Lincoln Co.**

of the Columbia River. The total drainage area impounded by the tailings dam is a 9.4 square miles. The dam is rated as large in size, and is classified as having a high (Category 1) downstream hazard potential (Foster, 1981). The high hazard ranking is attributed to the presence downstream of Highway 37 and the vermiculite product storage and shipping terminal located between the highway and the Kootenai River.

Existing outiets from the impoundment consist of a decant tower and a chute spillway constructed of half-sections of 8 foot diameter cormgated metal pipe (CMP). Normal flows from Rainy Creek are currentiy diverted around the impoundment through a CMP pipe constructed of 48 and 52 inch diameter sections, re-entering the original chamel approximately 800 feet downstream of the dam. All existing outiet and diversion structures will be removed as part of final closure.

The geology of the site consists of late Precambrian Belt Group consisting of fine-grained clastic and carbonate rocks which have undergone various degrees of metamorphism, and are covered with glacial outwash and till (Boettcher, 1963). The tailings impoundment is located on an intrusive rock body called tiie Rainy Creek stock, of which Vermiculite Mountain and W.R. Grace's mining area is a part. Depths to bedrock range from less than 2 feet to about 25 feet on the valley walls, and from 20 to 45 feet on the valley floor. Portions of the bedrock are weathered with low strength (Lewis, 1971).

The dam is located in Seismic Zone 2, with a potential for moderate earthquake damage. A study completed by Harding Lawson Associates (Vahdani, 1992) indicates "...*the dam is expected to remain stable during and following the design earthquake*", and "...... *results of our stability analysis indicate that the dam is stable during both static and dynamic loading conditions*".

Vegetation at the site consists of grasses, coniferous shrubs, and of mixture of deciduous (primarily cottonwood, alder, and aspen) and evergreen trees (cedar, larch, Douglas fir, ponderosa and lodgepole pine, and spruce). Active logging is taking place within the drainage basin, both on mine property and on adjacent Forest Service land. The tailings impoundment is currently devoid of vegetation.

2.3 SITE HISTORY/BACKGROUND

Vermiculite Mountain has long been the subject of mineral exploration because of the unique geology of the area. However, vermiculite production has been the only economically viable operation there. Mining was done as early 1890 but the first large scale activity was begun by the Zonolite Company beginning in the mid 1920's. W. R. Grace acquired the Zonolite Company in 1963 which continued to operate as the Zonolite Division of W.R. Grace. The first beneficiation process used an air separation method to process ore into a high grade vermiculite product. This process tended to produce high dust levels which took on increased significance with the recognition that asbestiform fibers could lead to certain kinds of lung disease. The ore body has occurrences of tremolite which is classified as an asbestos-like mineral. The process was converted to a wet process to reduce dust production during processing.

In 1971 W. R. Grace undertook a major expansion to increase capacity and improve the beneficiation process. It was at this time that the tailings impoundment was built to provide for settlement of the fine tails produced by the new process and to recover water for reuse (Foster, 1981; Boettcher, 1963; and Lewis, 1971). The tailings dam was designed by Bovay Engineers, Inc. of Spokane, Washington, and Harding Lawson Associates of Novato, California. The dam was designed and constructed in stages, with the 50 foot high (elevation 2830) starter dam constructed in 1971, immediately downstream of an older, existing dam. Additional construction phases in 1975, 1977, and 1980 have raised the top of dam elevation to 2925, for a total height of 135 feet measured from the downstream toe.

At the peak of operations, ore was processed at the rate of approximately 2,000,000 tons per year. Declining market conditions forced a gradual reduction in plant production from over 200,000 tons per year of product to less than 100,000 tons per year recently. In the fall of 1990 a decision was made to permanently close the facility because of the declining markets. Since 1990, the tailings impoundment has not received fine tails directly from the operations. However, small amounts of tailings from adjacent coarse tailings disposal areas continue to enter the reservoir through natural erosion processes, primarily surface runoff. These processes will be reduced as reclamation and reseeding efforts provide surface cover and stabilize the area.

A reclamation plan was submitted at the time of the expansion. However, the plan was very general and did not define or investigate specific actions in detail. One of the provisions of the permit was to provide for diversion of streams around mining wastes at the time of closure. In the case of the tailings impoundment, the requirements for diversion of a massive storm is calculated to be several thousand cubic feet per second. Our investigation of designs for successfully handling such a large quantity of water has suggested that other alternatives, using the storage capacity of the tailings impoundment might provide a safer and more effective resolution of this problem. The reasons for this conclusion are discussed in the sections which follow.

3.0 HYDROLOGIC EVALUATION

3.1 HYDROLOGIC PARAMETERS

In order to properly assess the requirements of the final closure design for the tailings impoundment it is necessary to evaluate the magnitude of streamflows for various levels of probability. We have analyzed three storm events here. A 10 year thunderstorm event was chosen to represent a condition which might be encountered on a regular basis and which might also be considered as a design parameter for some diversion altemates. A 100 year thunderstorm event was selected principally as the preferred basis for design of a partial diversion alternate, an event which would be exceeded only rarely thereby requiring use of emergency provisions on an infrequent time interval. A runoff equivalent to 0.5 of the probable maximum flood (PMF) event was also selected since the requirements for dam safety are based on the PMF and this value met or exceeded those requirements. There is also a recorded event in the area of a 0.5 PMF event. This event was a three day general storm; our analysis is based on a 6 hour thunderstorm event which produces a more intense runoff in a drainage of this size. The methodology for calculation of these design storms is described in Section 3.2.

The W.R. Grace tailings dam is located on Rainy Creek, approximately 2000 feet below the confluence of Rainy and Fleetwood Creeks. The dam impounds 9.4 square miles (sq. mi.) of the Rainy Creek drainage basin, of which 5.9 sq. mi. is drained by Rainy Creek, and 3.5 sq. mi. is drained by Fleetwood Creek. The two flows enter the impoundment from the north and east, respectively. The drainage basin is generally "L" shaped above the dam (Figure 3.1). Average stream gradients for Rainy and Fleetwood Creeks are 12.2% and 11.1% respectively.

The Rainy Creek drainage basin is located on a southern exposure of the Purcell Mountains, and is primarily forest covered except for the area disturbed by the mining/milling operations and logging operations. The basin rises from an elevation of approximately 2900 at the surface of the tailings impoundment, to 6040 feet at the top of Blue Mountain. The longest length of channel is about 4.9 miles for Rainy Creek, and about 3.1 miles for Fleetwood Creek. Average channel slopes are 5 to 15 percent, with sideslopes ranging from 5 to 45 percent. Rainy Creek enters the Kootenai River approximately 2 1/2 miles downstream of the tailings dam.

Mean annual precipitation at Libby is 19.4 inches, with 37 percent of it occurring in the months of November through January, and 18 percent falling in the months of May and June. The month having the highest average precipitation is January with 2.42 inches.

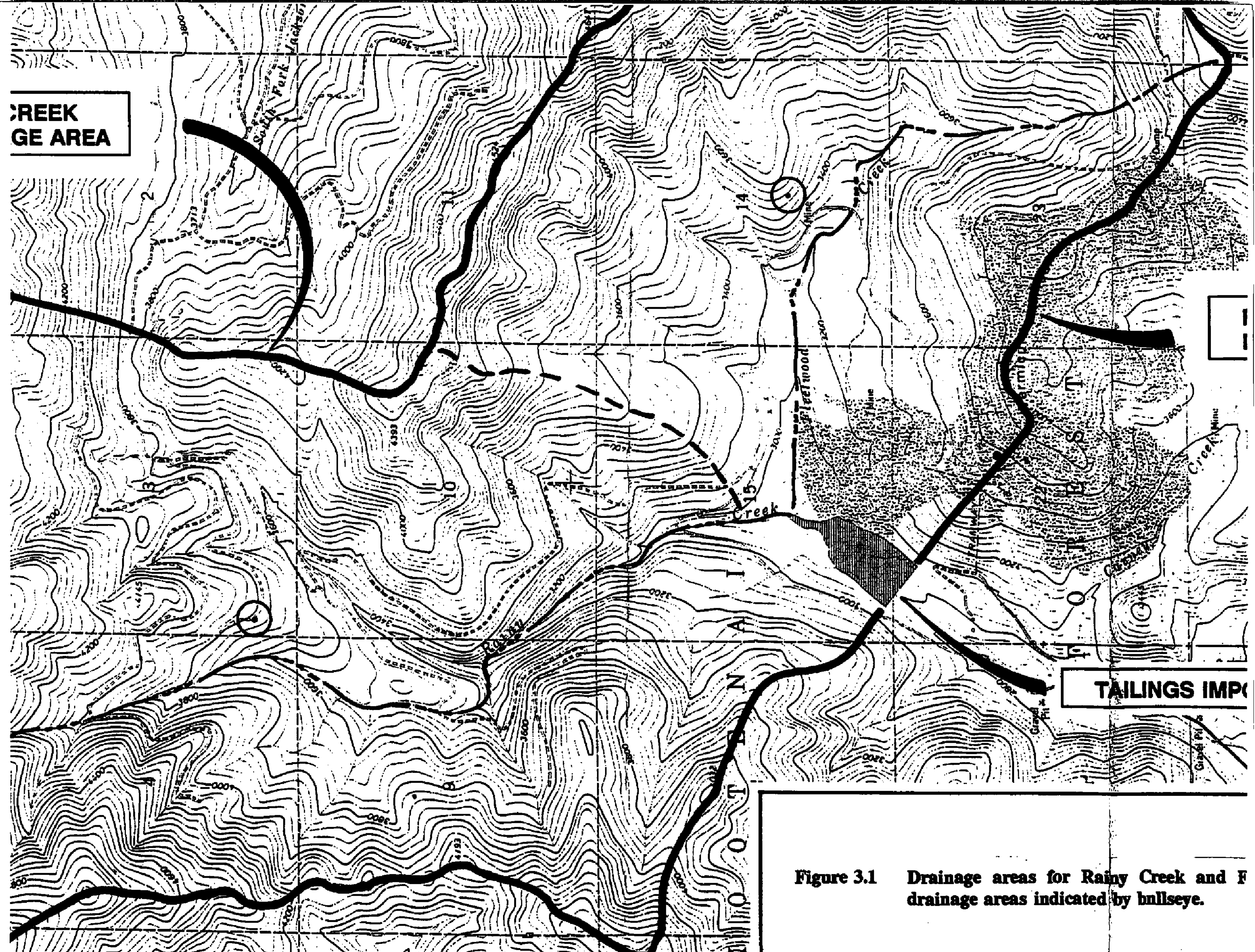


Figure 3.1 Drainage areas for Rainy Creek and F drainage areas indicated by bullseye.

Temperature in Libby ranges from an average of 22.4° Fahrenheit (F) in January to an average of 67°F in July. Average annual precipitation at the site is estimated at 30 inches per year (USDA, 1977), and the temperature would be expected to average 3 to 5 degrees cooler than at Libby. Climatological data was obtained from the Libby 1 N.E. Ranger station.

Soils in the area have been assigned a Hydrologic Soil Classification of "B" by the Soil Conservation Service (SCS). The drainage basin is estimated to have >75% ground cover of mature forest in good condition, with moderate slopes. Antecedent moisture is considered to be average. A "Curve Number" of 60 is estimated for both the Rainy Creek drainage basin and the Fleetwood Creek drainage basin. As discussed in Section 3.2, Curve Numbers are used in the SCS hydrologic model to classify the drainage characteristics of different terrains. To assure a conservative runoff estimate, the curve number was selected slightly higher than normally recommended for forested lands to account for the impact from mining on areas of the Fleetwood Creek drainage and extensive clear cuts in Upper Rainy Creek. A summary of design conditions is shown in Table 3.1.

Table 3.1. Hydrologic parameters for Rainy Creek and Fleetwood Creek drainage areas impounded by the tailings dam.

WATERSHED NAME	AREA (sq. miles)	SCS CURVE NUMBER	AVE. SLOPE (%)	CHANNEL LENGTH (ft)	SOIL GROUP
Rainy Creek	5.9	60	12.2	25,870	B
Fleetwood Creek	3.5	60	11.1	16,370	B

3.2 DESIGN STORMS

Runoff from three design storms was used to evaluate flood routing through the tailings impoundment, specifically 1) a 10-year frequency, 24-hour precipitation event; 2) a 100-year frequency, 24-hour precipitation event; and, 3) a 6-hour probable maximum flood (PMF).

A spreadsheet program developed by Schafer and Associates was used to simulate the runoff from the 10 year and 100 year, 24 hour precipitation events. The model uses the calculation procedures outlined in the SCS National Engineering Handbook, Section 4, Hydrology (NEH-4). The SCS method finds a watershed flow hydrograph using the "Curve Number" method. A complete description of the background, methods and procedures is given in NEH-4 (U.S. Dept. of Agriculture, 1985). A brief description is provided below.

The SCS Curve Number Method was developed for areas having little rainfall data, particularly for storm duration and intensity. Runoff does not begin until after some period of "initial abstraction" (la) where infiltration, interception, and surface storage occur. The la is estimated to be 20 percent of the maximum potential runoff. Rainfall-rnnoff relations, based on SCS curve numbers, are then developed to estimate the runoff volume and timing from a precipitation event.

Curve numbers are selected based on land use, soil type, cover, hydrologic condition and antecedent moisture (see Section 3.1). Other necessary information includes average slope, drainage area and longest runoff length, and rainfall distributions as a SCS Type II convective thunderstorm event. Lag time, time of concentration, time to peak, etc. are calculated from the curve numbers. A series of elemental hydrographs, based on peak flows and the values of the dimensionless unit hydrograph (SCS), are developed for each duration, which in turn are summed to produce a total hydrograph. See Sections 3.2.1 and 3.2.2.

The PMF was calculated using the method outlined in the Department of Interior, Flood Hydrology Manual (U.S. Dept. of the Interior, 1989). The method is based on development of a "Synthetic Unit Hydrograph" which is used to estimate surface runoff from probable maximum precipitation. A brief description is given in section 3.2.3.

3.2.1 10-Year Event

A 10-year, 24-hour antecedent storm precipitation of 2.4 inches for Rainy Creek drainage basin was obtained from the National Oceanic and Atmospheric Administration (NOAA) Atias (U.S. Dept. of Commerce, 1973). Using this precipitation value, and the boundary conditions outlined in Sections 3.1, a peak runoff for Rainy Creek (65 cfs) occurred 16.3 hours after the beginning of the storm. Peak runoff for Fleetwood Creek (45 cfs) occurred at 14.9 hours. Model results for the runoff of each drainage area are found in Appendix A. Key parameters for this model are summarized in Table 3.2. Figure 3.2 is a graphical representation of the surface water runoff and rainfall intensity for a 10-year, 24-hour event.

The total runoff hydrograph for the entire watershed area unpounded by the tailings dam was obtained by summing the two individual hydrographs, resulting in a peak flow of about 107 cfs occurring at 15.5 hours after the beginning of the event. The total runoff for the affected drainage area is 74 acre-ft, with 46 acre-ft from Rainy Creek, and 28 acre-ft from Fleetwood Creek.

3.2.2 100-Year Event

A 100-year, 24-hour antecedent storm precipitation of 3.4 inches was obtained from the NOAA Atlas (U.S. Dept. of Commerce, 1973). Using this precipitation value, and the boundary conditions outlined in Sections 3.1, a peak runoff for Rainy Creek (262 cfs) occurred 15.2 hours after the beginning of the storm. Peak runoff for Fleetwood Creek (204 cfs) occurred at 14.4 hours as summarized in Table 3.3. Model results for the runoff of each drainage area are found in Appendix A. Figure 3.3 shows the surface water runoff and rainfall intensity for a 100-year, 24-hour event.

Table 3.2. **Surface water runoff for a 10-year, 24-hour precipitation event using SCS Type II rainfall distribution.**

10-YEAR, 24-HOUR STORM EVENT				
WATERSHED NAME	PRECIPITATION (inches)	RUNOFF (inches)	PEAK FLOW (cfs)	TIME OF PEAK (hrs.)
Rainy Creek	2.4	0.147	65	16.3
Fleetwood Creek	2.4	0.147	45	14.9
Combined Flows	2.4	0.147	107	15.5

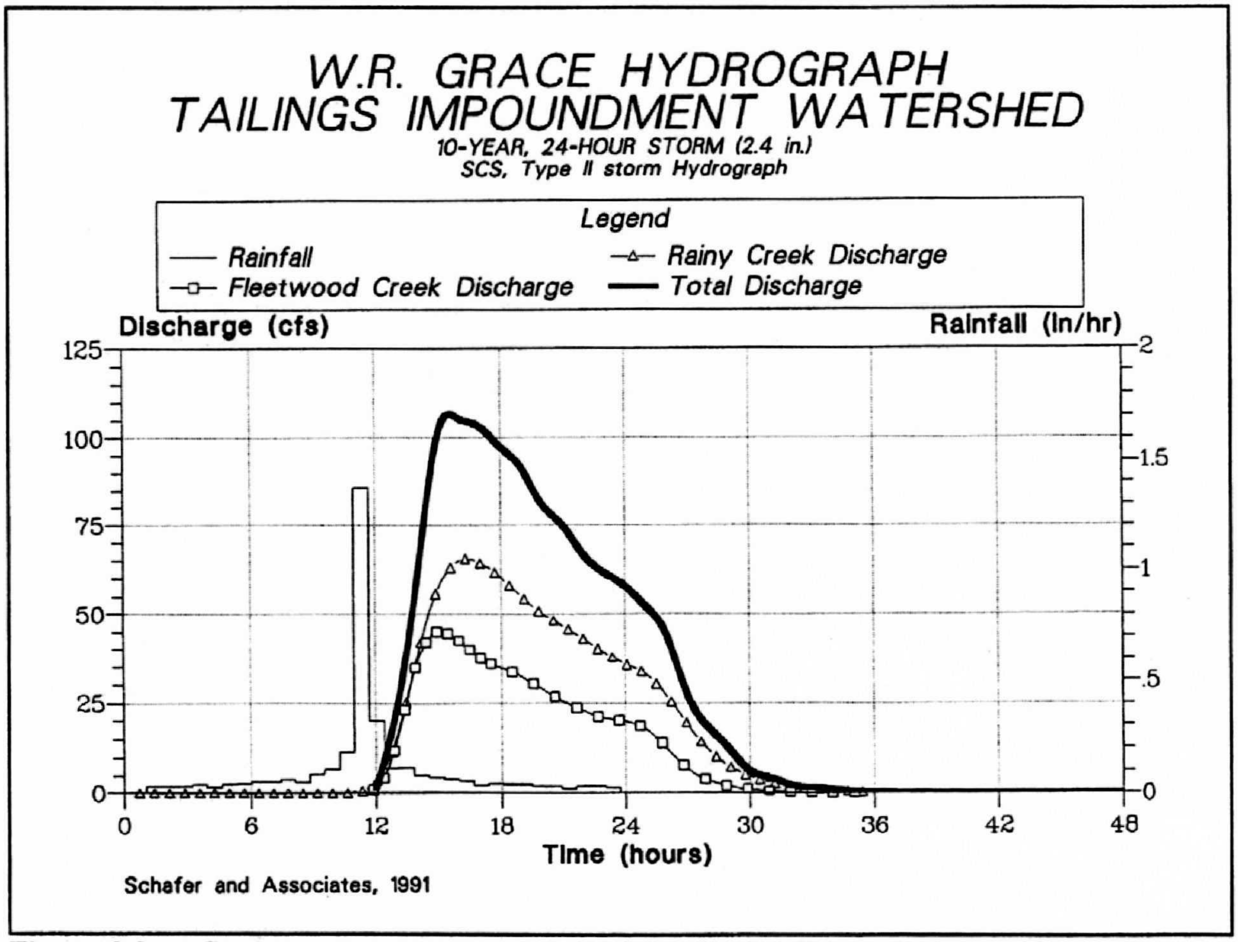


Figure 3.2 **Surface water runoff hydrographs and rainfall intensity for a 10-year, 24-hour storm (2.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

Table 3.3. **Surface water runoff for a 100-year, 24-hour precipitation event using SCS Type II rainfall distribution.**

100-YEAR, 24-HOUR STORM EVENT				
WATERSHED NAME	PRECIPITATION (inches)	RUNOFF (inches)	PEAK FLOW (cfs)	TIME OF PEAK (hrs.)
Rainy Creek	3.4	0.489	262	15.2
Fleetwood Creek	3.4	0.489	204	14.4
Combined Flows	3.4	0.489	460	14.8

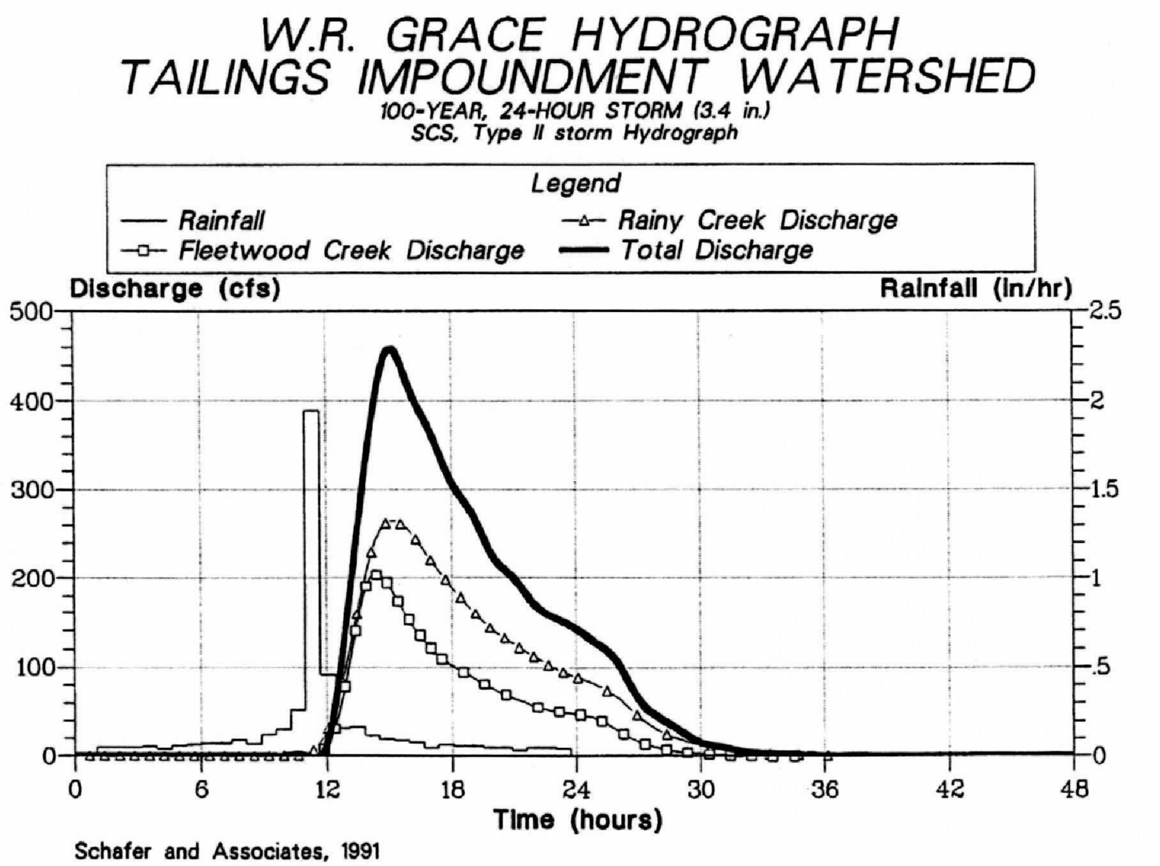


Figure 3.3 **Surface water runoff hydrographs and rainfall intensity for a 100-year, 24-hour storm (3.4 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

The total runoff hydrograph for the entire watershed area impounded by the tailings dam was obtained by summing the two individual hydrographs, resulting in a peak flow of 460 cfs occurring at 14.8 hours after the beginning of the event (Fig. 3.3). The total runoff for the drainage area is 245 acre-ft, with 154 acre-ft from Rainy Creek, and 91 acre-ft from Fleetwood Creek.

3.2.3 Probable Maximum Flood

The probable maximum flood (PMF) is the flood expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a region. Three scenarios are most often considered when estimating the PMF, specifically 1) general seasonal storms (October through June), 2) rain on snow (including snowmelt) and, 3) summer convective thunderstorms. Based on the Hydrometeorological Report No. 43 (HMR 43), "Probable Maximum Precipitation, Northwest States" (U.S. Weather Bureau, 1966), intense local summer thunderstorms of short duration are most likely to produce a PMF event in this region of the United States (east of the Cascade divide and west of the Rocky Mountains).

Using the method outlined in HMR 43 for summer thunderstorms in small drainage basins (<550 square miles), a PMF event is estimated to produce 10.7 inches of precipitation in 6 hours, distributed as shown by the hyetograph in Figure 3.4. Detailed calculations used to determine the PMF hyetograph are located in Appendix B.

Runoff from the PMF is calculated using the method outiined in the Bureau of Reclamation "Flood Hydrology Manual" (U.S. Dept. of the Interior, 1989). This method is similar to the SCS method described in Section 3.1, with the exception of the runoff determined by a synthetic unit hydrograph instead of summing a series of dimensionless unit hydrographs (SCS method). Input data requirements are similar, including drainage area, channel length, average slope, and ultimate infiltration (based on the SCS hydrologic soil group). As in the SCS method, lag time, duration, and incremental runoff are calculated from the input data. Input conditions are similar to those found in Section 3.1, with the exception of antecedent moisture conditions considered to be near or at saturation.

Important runoff parameters for this event are summarized in Table 3.5. The peak runoff for a PMF event in the Rainy Creek drainage area was calculated to be 7330 cfs, occurring 5.5 hours after the beginning of the storm. Peak runoff for Fleetwood Creek was calculated at 5884 cfs occurring at 4.5 hours after the beginning of the storm. Detailed calculations of the PMF runoff are located in Appendix B.

The total PMF runoff hydrograph for the entire watershed area impounded by the tailings dam was obtained by summing the two individual hydrographs (Rainy and Fleetwood Creeks), resulting in a peak flow of 11,676 cfs occurring at 5.0 hours after the beginning of the storm event (Figure 3.5). The total runoff for the drainage area is 4612 acre-ft, with 2895 acre-ft from Rainy Creek, and 1717 acre-ft from Fleetwood Creek.

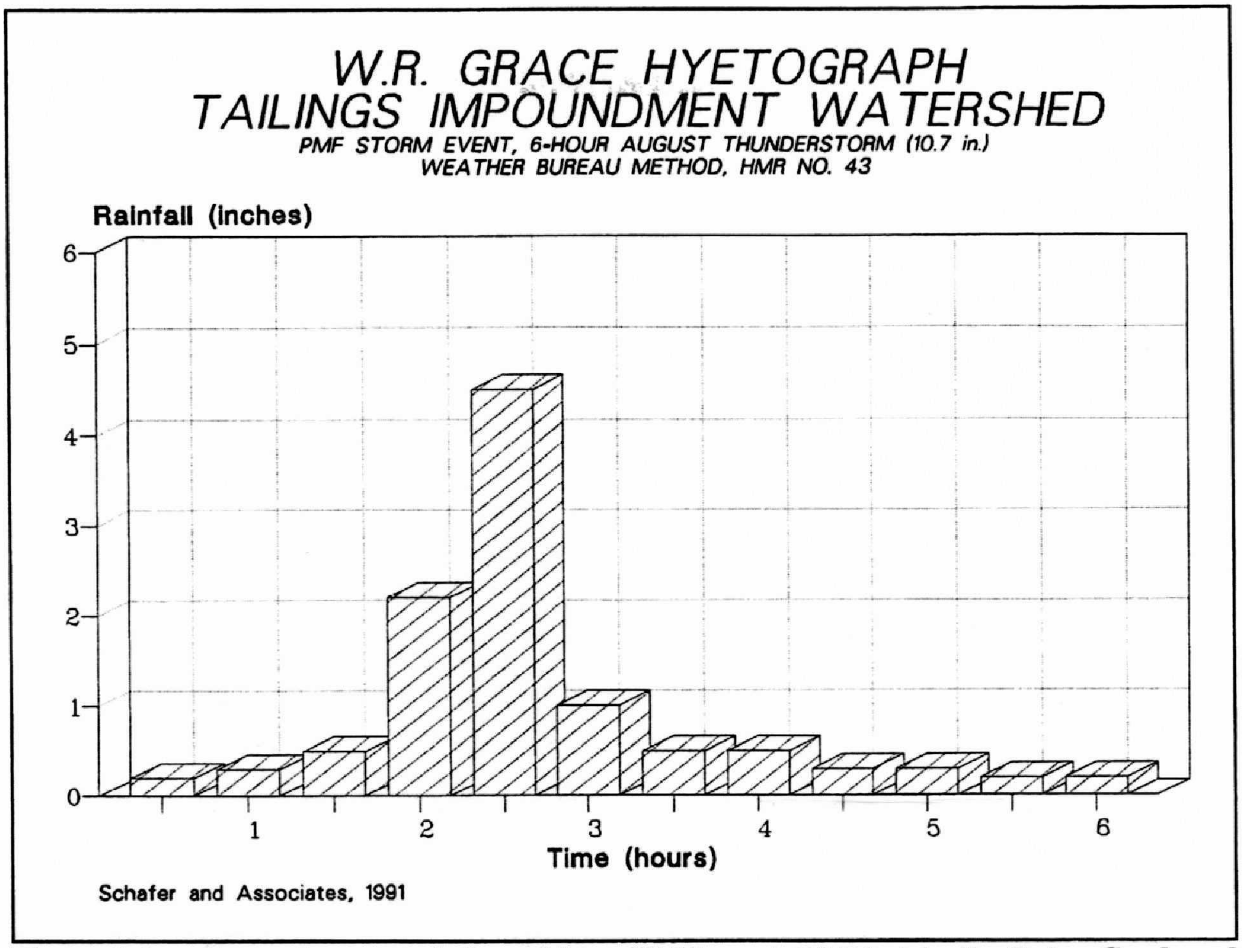


Figure 3.4 **Storm hyetograph for a 6-hour PMF event (10.7 in.) in the Rainy Creek and Fleetwood Creek drainage basins.**

Table 3.4. **Surface water runoff for a 6-hour PMF event (10.7 in.) using the storm distribution hyetograph of Figure 3.4.**

PROBABLE MAXIMUM FLOOD (6-HOUR)				
WATERSHED NAME	**PRECIPITATION (inches)**	**RUNOFF (inches)**	**PEAK FLOW (cfs)**	**TIME OF PEAK (hrs.)**
Rainy Creek	10.7	9.20	7330	5.5
Fleetwood	10.7	9.20	5884	4.5
Combined Flows	----	----	11676	5.0

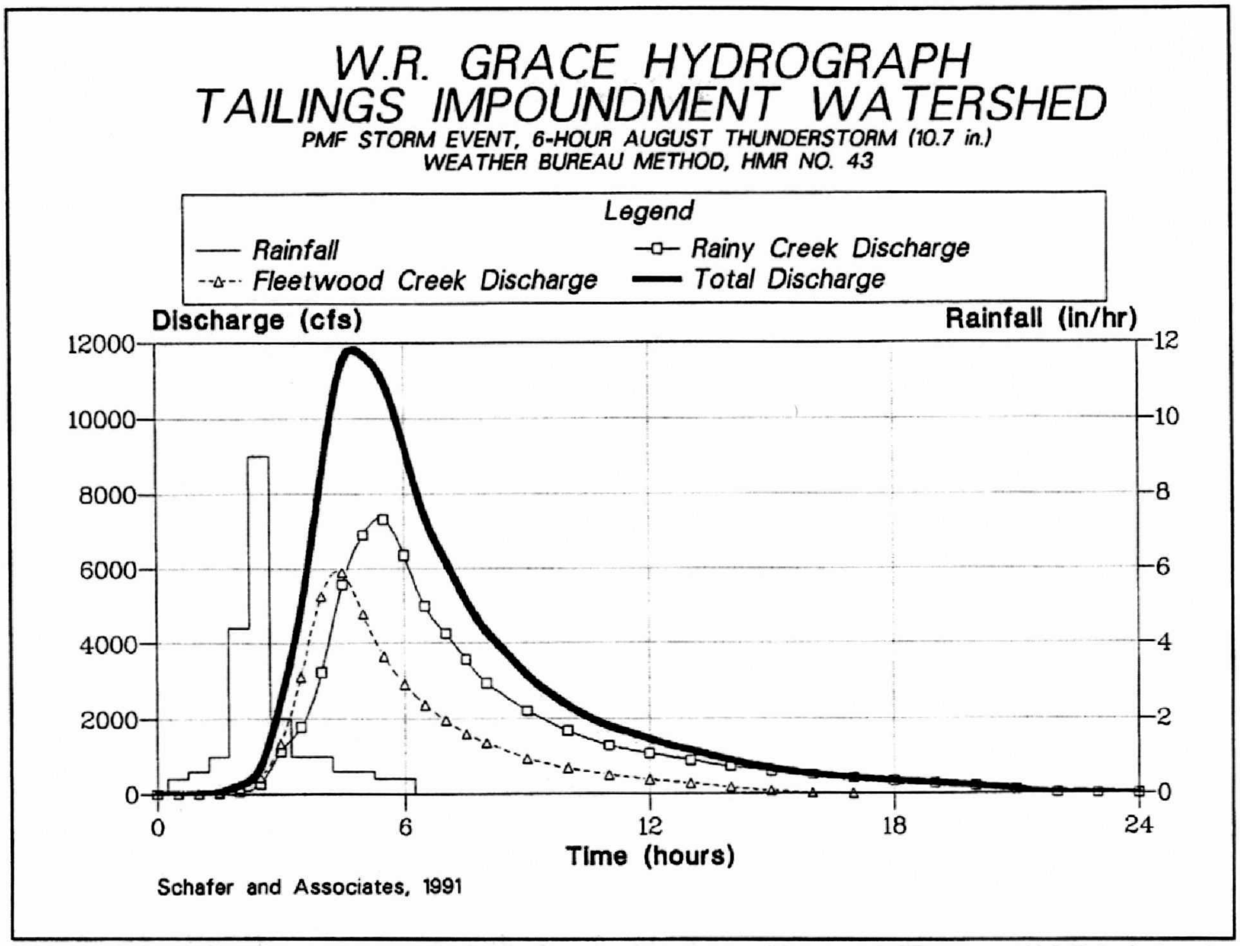


Figure 3.5 **Surface water runoff hydrographs for a 6-hour PMF event (10.7 in.) in the Rainy Creek and Fleetwood Creek watersheds.**

3.3 TAILINGS IMOUNDMENT CAPACITY

The top of dam elevation of the vermiculite tailings dam is 2926, with an existing emergency spillway crest elevation of 2920. The top of tailings elevations range from a low of 2895 just north of the decant tower, to a high of 2914 at the southeast corner of the impoundment. Average tailings elevation is estimated to be slightly over 2900.

Using the conic (volume) method to determine the reservoir storage capacity, it is estimated that the reservoir will have a surface area of 68.7 acres and a storage volume of 871 acre-feet measured to the crest of the (existing) emergency spillway. Approximately 431 acre-feet of storage is available between the existing emergency spillway crest and the dam crest, making the total storage capacity (top of dam) 1302 acre feet. A tabulation of impoundment capacities as a function of elevation is given in Table 3.5.

Table 3.5. Storage capacity of the tailings impoundment/reservoir.

ELEVATION (ft.)	AREA (acres)	INCREMENTAL VOLUME (acre-ft.)	CUMULATIVE VOLUME (acre-ft.)
2895[1]	-	-	-
2900	10.4	26.0	26.0
2905	21.0	78.5	104.5
2910	48.7	174.3	278.8
2915	59.7	271.0	549.8
2920[2]	68.7	321.0	870.8
2926[3]	74.9	430.8	1301.6

1 Lower limit of impoundment.
2 Emergency spillway crest elevation.
3 Top of dam elevation.

During the closure work on the impoundment, it is proposed that the existing emergency spillway will be removed, and a new emergency spillway constructed on the west side of the dam. The emergency spillway will work in conjunction with a proposed primary outlet/control structure to route flows through the reservoir. See Section 5.0 for details of the preferred alternative.

3.4 DAM SAFETY REQUIREMENTS

The Rainy Creek Basin Zonolite Tailings Dam, MT-1470 has been rated as large in size and as having a high downstream hazard potential (Category 1), as determined by an inspection and report completed by Morrison-Maierle in 1981 (Foster, 1981). The inspection was conducted in accordance with U.S. Army Corp of Engineers Guidelines for Safety Inspection of Dams, and was completed for the State of Montana Department of Natural Resources and Conservation, under Public Law 92-367. The classification is based on a dam height of 135 feet, and storage capacity of 2120 acre-feet at the spillway crest.

Under State of Montana regulations for Dam Safety, Rule 36.14.206 (State of Montana, 1989):

(1), *".....hazard determination shall be based on the consequences of dam failure--not the condition, probability, or risk of failure. A dam must be classified high-hazard if the impoundment capacity is 50 acre-feet or larger and it is determined that a loss of human life is likely to occur within the breach flooded area as a result of failure of the dam."*

(3) " ..... *the effects of flood inundation...... will continue downstream until the flood stage is equal to that of the 100 year floodplain.*", and

(5) "*Loss of life is assumed to occur if the following structures are present: .... other paved highways....*".

Under Rule 36.14.502:

(1) "*Spillways (principal and emergency) for high-hazard dams must safely pass the flood calculated from the inflow design flood. The minimum inflow design flood is expressed as a fraction of the probable maximum flood or as otherwise indicated in Table A*" (See Table 3.6),

(2) "...... *The minimum inflow design flood shall be the 100-year, 24-hour flood*",

(3) "......... *routing of the inflow design flood through the reservoir shall assume storage contents to be at the emergency crest elevation prior to routing*",

(4) "....*breach area ..... is designated as Category A if .... major repair or alteration of the emergency spillway is to be performed, where the downstream hazard contains more than 20 residences and the failure flood wave is less than 4 hours from the dam to the first residence*",

(5) ".....*breach area ...... is designated as Category B if the dam is an existing dam not meeting the criteria for a Category A dam*".

Table 3.6. Emergency spillway inflow design flood(s) from Table A of the Montana Dam Safety regulations, Rule 36.14.502.

CAPACITY TO THE EMERGENCY CREST/HEIGHT TO DAM CREST	BREACH AREA CATEGORY A	BREACH AREA CATEGORY B
Dams less than 100 acre-feet and less than 20 feet in height	2Q	Q
Dams less than 500 acre-feet and less than 35 feet in height	.2 PMF	.1 PMF
Dams less than 1000 acre-feet and less than 50 feet in height	.3 PMF	.15 PMF
Dams less than 12,500 acre-feet and less than 50 feet in height	.5 PMF	.5 PMF
Dams less than 50,000 acre-feet and less than 100 feet in height	.75 PMF	.75 PMF
Dams 50,000 acre-feet or greater and 100 feet or greater in height	1.0 PMF	1.0 PMF

We applied the following conditions to select design criteria for the project:

- with the top of tailings elevation of 2900+, the height to the crest of the dam (from the tailings surface) is less than 50 feet;
- the capacity of the reservoir to the existing emergency spillway crest is less than 1000 acre-feet;
- there are no residences between the dam and the Kootenai River, however, a paved highway does exist;
- the impact to the Kootenai River of a dam breach is unknown, but is not expected to exceed the 100-year floodplain at the closest residence downstream;
- the work will be considered to be a major alteration to an existing dam.

Based on these criteria and the provisions in statutes cited above, the tailings dam is considered to be high-hazard, making it applicable to all other criteria for high-hazard dams. The breach area below the dam is unknown, therefore it will be considered as Category A. Based on these guidelines and criteria, the required design flow in Table 3.6 is 0.30 PMF, or 3504 cfs.

The flood routing volume proposed by W.R. Grace is 0.5 PMF, which calculates to a design value of 5838 cfs (0.5 x 11,676 = 5838). This 0.5 PMF value will be used during flood routing analyses.

3.5 PROPOSED DESIGN FLOWS

W.R. Grace proposes to use the flows summarized in Table 3.7 for flood routing through the vermiculite tailings impoundment. Boundary conditions and assumptions follow:

- A 2.4 inch, 24 hour design storm to simulate a 10-year return storm; and 3.4 inch, 24 hour design storm to simulate a 100-year return storm. Both storms are distributed as a SCS Type II convective thunderstorms;
- A 10.7 inch, 6 hour design storm to simulate a probable maximmn flood (PMF) event, distributed as a convective thunderstorm according to U.S. Weather Bureau guidelines;
- Soils within the drainage classify as SCS type "B" soil group. The soils contain average in-situ antecedent moisture for the 10 year and 100 year return storms. Soils are considered to be near saturation, with 0.25 inch per hour infiltration for PMF event;

- The drainage basins are dense forest in good condition, with >75% ground cover;

- Curve numbers of 60 are used for both Rainy Creek and Fleetwood Creek drainage basins.

- The tailings dam is classified as a high-hazard dam according to Montana Dam Safety, and U.S. Army Corp of Engineers regulations;

- The required inflow design is 0.30 PMF, based on less than 50 foot dam height (from surface of tailings), less than 1000 acre-feet storage at emergency spillway crest, and a Category A breach area (State of Montana, 1989);

- 0.5 PMF will be used for flood routing analyses and design;

- The existing tailings impoundment decant tower and emergency spillway, and the Rainy Creek diversion and pipeline will be removed during closure.

Table 3.7. Design flood volumes proposed for flood routing alternatives analysis and conceptual design.

WATERSHED NAME	10-YEAR, 24-HOUR (cfs)	100-YEAR, 24-HOUR (cfs)	0.5 PMF 6-HOUR (cfs)
Rainy Creek	65	262	3665
Fleetwood Creek	45	203	2942
Combined Flows	107	460	5838

4.0 FLOOD ROUTING

4.1 OVERVIEW OF ALTERNATIVES

The project calls for engineering analysis of available alternatives for routing floods through the area affected by the vermiculite tailings impoundment. Concerns that will be addressed by the analysis include safety, potential for water contamination especially from asbestiform fibers, long-term stability of the impoundment including an analysis of tailings dam saturation and seismic events, sedimentation, and others concerns.

Three basic options for flood routing have been considered: Alternate I - diverting all flows, including storms producing PMF events, around the impoundment and dam, Alternate 11 - routing flows through the impoundment and discharging through an outiet channel constructed in or near the dam and Alternate III - a partial diversion of "normal" stream flows and routing of events exceeding diversion design flows into the impoundment. Flood routings were modeled using a computer program entitied "Hydrograph Develop Program", developed by the SCS in 1990. Routing models were completed by Lew Burton and Ed Juvan, retired SCS engineers.

Within each of the general alternates are several design variations which have been considered in varying degrees of detail. Table 4.1 provides a summary of the pertinent features of each option considered. A discussion and evaluation of the alternatives follows in Sections 4.2 through 4.4. A description of design details for the preferred alternative is given in Section 5.0.

In the following investigations, each main alternative will begin with a discussion of general parameters, followed by specific routing alternatives, and finally a summary of advantages and disadvantages. Maps, sections, and other design drawings will be provided as necessary. The project area has been set up as a grid, with the north-south (horizontal) axis designated by letters (A - L), and the east-west (vertical) axis designated by numbers (1 - 9). This should provide for a more efficient method of locating sections or more detailed drawings. The base grid system is delineated on Plate 1.

Table 4.1 Summary of alternatives considered for flood routing.

Alternative	Essential Design Features
Full Diversion **Alternate Ia:** Partial Isolation of Tailings **Alternate Ib:** Total Isolation of Tailings **Alternate Ic:** West Side Diversion Channel **Alternate Id:** East Side Diversion Channel **Alternate Ie:** Pipeline	• Diversion dam(s) upstream of tailings dam to intercept streams • Flood routing in large channels around dam • Large drop chutes for return of stream flow to Rainy Creek below dam
Channel Reconstruction in Tailings **Alternate IIa:** Water Level at 2904' **Alternate IIb:** Water Level at 2910' **Alternate IIc:** East Abutment Outlet **Alternate IId:** West Abutment Outlet **Alternate IIe:** Outlet Over Dam Face	• Streams enter impoundment and collect in a pond at the upper end with water level kept away from dam for improved stability • Unused tailings impoundment capacity used for storm surge up to 0.5 PMF • Lined channel (for erosion control) delivers water to outlet structure at the dam • Box culvert outlet control structure reduces stream discharge from impoundment during major storm events • Optional emergency spillway for storms in excess of 0.5 PMF • Armored channel/drop structures return stream flow to Rainy Creek below the dam
Partial Diversion **Alternate IIIa:** 100-Year Storm Diversion **Alternate IIIb:** 10-Year Storm Diversion	• Diversion dam(s) upstream of tailings dam intercepts Rainy and Fleetwood Creeks • Outlet control structure reduces stream discharge from diversion dams to a design maximum which is routed around the tailings • Drop chutes similar to Alternate I but smaller return diverted stream flow to Rainy Creek below tailings dam • Runoff in excess of design maximum overflows to the tailings impoundment • Secondary outlet and discharge channel similar to that of Alternate II

4.2 FULL DIVERSION ALTERNATIVES

4.2.1 Description of Design Concepts

<u>Common Diversion Dam (Alternate Ia):</u> Diversion of Rainy and Fleetwood Creeks around the impoundment is one possible method of flood routing following closure. Full diversion will entail intercepting, diverting both creeks around the impoundment, and ultimately returning them to Rainy Creek downstream of the dam.

Construction of a diversion dam across the upper end of the existing impoundment would be required at a location where flows from Rainy Creek and Fleetwood Creek join. The flows would then be diverted around the tailings impoundment through an open channel or pipe constructed adjacent to the impoundment. Once past the dam, a concrete drop chute or other means of elevation reduction would return the diverted flows to Rainy Creek. Plate 2 is a conceptual plan view of this alternate.

A full diversion dam, capable of diverting a 0.5 PMF event while retaining long-term structural integrity, will be very difficult to construct because of the tailings in the impoundment and east abutment. Tailings will not provide a competent foundation for the dam base or abutment, hence significant excavation of the tailings would be required (see Plate 3). Conventional construction methods and equipment often fail when working in tailings, making the project costly and with questionable results.

<u>Separate Diversion Dams (Alternate Ib):</u> An alternative would be to construct a diversion dam at the extreme upper end of the impoundment, beyond the extent of the tailings. A separate diversion dam would be constructed for Fleetwood Creek upstream of the coarse tailings dump. Flows from Fleetwood Creek would be delivered to the Rainy Creek diversion by a constructed channel (Plate 4-A). Both flows would then enter a main diversion channel and be routed around the impoundment as above (Plate 4-B)

<u>West Side Channel (Alternate Ic):</u> Should full diversion be selected, the best method for carrying the diverted flows around the tailings impoundment would be an open channel constructed on the west side of the impoundment. The channel would be constructed in natural material (off the tailings), and connected to a concrete drop chute/plunge pool below the tailings dam. Flows would be diverted into the constructed channel at the diversion dam, carried around the tailings dam and impoundment, and returned to Rainy Creek downstream of the dam. Refer to Plates 2, 4-A, and 4-B.

A conceptual design was completed for a 0.5 PMF channel on the west side of the tailings using a beginning channel elevation of 2900.0, and a gradient of 0.005 ft/ft (0.5%). The structure would be a rock-lined, trapezoidal open channel with 20 ft wide (flat) bottom and 2:1 sideslopes. With a design flow of 0.5 PMF (5838 cfs) and applying Manning's Equation:

$$Q = A\frac{1.486}{n}R^{2/3}S^{1/2}$$

in which:

Q = volume of flow, cfs
A = cross-sectional area of flow in ft^2
S = slope, ft/ft
R = hydraulic radius, ft
n = coefficient of roughness (0.04 for rock lined channels)

a peak flow depth of about 12 feet is calculated with a velocity of approximately 11 feet per second. With the beginning channel elevation of 2900 and 0.005 ft/ft gradient, the bottom elevation of the channel opposite the dam will be about 2888. Recommended maximmn cut slopes are 2:1, witii spaced 10 ft safety benches where possible. The chaımel would be armored with a minimum of 24 inches of D_{50} = 18 inch-rock liüing to handle the velocities associated with peak flows corresponding to the predicted peak water level. Plate 5 shows a typical cross-section of the west side diversion channel (relative location shown on Plate 2).

East Side Channel (Alternate Id): An alternate full diversion channel would be to construct an open chaımel on the east side of the impoundment. The channel would be similar to tiie west side with a concrete drop chute/plımge pool. Flows would be diverted into the channel at the diversion dam, carried around the impoundment, and returned to Rainy Creek downstream of the tailings dam.

This alternate is not practical due to the proximity of the coarse tailings dump, and presence of shallow bedrock and steep slopes. The beginning section of the chaımel would be located entirely within the coarse tailings durnp which is unconsolidated and geotechnically unstable. Significant design and engineering would be necessary to constrnct a channel in this material. Further, lining would be required to prevent rapid infiltration and increased foundation instability. Excavation to natural material would be virtually impossible.

On the lower sections of the channel, the depth to bedrock is generally less than 10 feet (Lewis, 1971) and portions of the drainage sideslopes are veıy steep. These restrictions, coupled with tiie required channel size for 0.5 PMF, would require that the channel be constrncted partially within the fine tailings (see Plate 6). An alternative would be to constmct the channel entirely in bedrock (see Plate 7), requiring extensive drilling and blasting. Either channel location has drawbacks.

Pipeiine (Alternate Ie): A pipeline, or other closed conduit, was explored as an alternate for carrying full diversion flows around the tailings impoundment. As with the open channels, the entire flow from both Rainy and Fleetwood Creeks would be diverted into the pipeline which would carry this flow around the impoumdment and return it to Rainy Creek downstream of the tailings dam. The pipeline would most likely be located on the west abutment, and would eliminate the need for a drop stmcture.

The primary advantage to a pipeline is the elimination of water loss through infiltration, and associated tailings dam saturation problems. Another advantage is the reduction in public accessibility, with the exception of the pipe entrance.

Disadvantages include size, geotechnical stability, maintenance, and cost. A pipeline greater than 20 ft (diameter) is required to carry 5838 cfs, the exact size depending on shape and material type. To properly install a pipe of this size requires extensive excavation, and specialized construction methods and equipment. Pre-stressed concrete pipe would be the best choice, but with considerable cost. Even with pre-stressed concrete, geotechnical stability may remain a problem, due primarily to the geology and topographic relief of the area.

A safety concern is the entrance into the pipeline, and the closed system preventing quick escape. Installation of a grate, or other barrier would prevent this, but would greatly increase maintenance and the possibility of plugging with subsequent system failure during major events.

4.2.2 Evaluation of the Full Diversion Alternatives

<u>Safety:</u> Safety and long-term integrity of any system are directly related, and should be the primary considerations when selecting a flood routing system. The full diversion alternate <u>increases</u> the potential for failure, and <u>decreases</u> the safety of the system. The drop chute and plunge pool, constructed of reinforced concrete, would be difficult to build on steep slopes such as these. Stability of the structure in a massive flood condition would be problematic.

The channels carrying the diverted flows would be very large, and inherently less stable than smaller channels, particularly when constructed into the side of a hill as they would be in this case. From a hydrologic and geotechnical standpoint, any channel, natural or constructed, located above the low point in a drainage is generally not considered to provide good long-term service, particularly when considering flows of this magnitude.

For the east side diversion channel, the combination of construction difficulties and doubtful foundation/geotechnical factors make this alternative a poor choice for a long-term diversion channel. For both east and west side channels, construction of the drop chute will be costly, and plugging during high flows a primary concern.

The drop chute below the tailings dam would be a large, concrete structure to handle the volume and velocity of the peak flows. Construction on the steep terrain of the west abutment area will be very expensive, and long-term geotechnical stability may be difficult to obtain. Other safety considerations include public accessibility to the large, fast moving flows in the channels and drop chute, and the difficulty in "escaping" from such.

The diversion dams are designed to only collect water prior to routing around the impoundment and would have little useful storage capacity. Should the diversion channels become plugged, or the system fail for some other reason, the flood flows would quickly breach the diversion dams and enter the impoundment. The breach could be rapid, in turn causing a large surge of water to strike the tailing dam. If the tailings dam did not fail from impact, the impoundment would begin to fill and could cut a new channel from the tailings impoundment into the diversion channel, or in an improbable event, could block the diversion channel with debris so badly that overtopping of the impoundment might occur. Either event would bring the potential for extensive uncontrolled erosion of the tailings material. Overtopping the dam could cause catastrophic failure of the dam unless additional precautions are taken. Dams in a series are not considered to be good engineering practice.

Full diversion of a 0.5 PMF event (producing 5838 cfs) requires a complex system of very large diversion dams, channels, and drop chute to route the entire peak flow of a storm of this magnitude around the impoundment, and return it to Rainy Creek downstream of the tailings dam. This alternate ignores the potential for flood control in the unused storage capacity of the impoundment. By allowing the reservoir to surge and temporarily store the peak flood flows, outflow peaks can be reduced to roughly 15 percent of the peak inflow (5838 cfs) and still contain a 0.5 PMF event.

Water Quality Impacts: While water contamination, particularly from tremolite fibers may be reduced by diversion, it will not be eliminated. Constructing a diversion dam to collect both flows simultaneously will include a section of the tailings impoundment. In addition, Fleetwood Creek will be flowing through the coarse tailings.

Asbestiform fiber contamination from the tailings impoundment and coarse tailings dump could be eliminated by the second diversion alternative shown in Plate 4-A and 4-B. This alternative would prevent streamflows from contacting the tailings, however, these fibers would continue to enter Fleetwood Creek from the natural vermiculite intrusive from which Fleetwood Creek originates. Further, Carney Creek, which enters Rainy Creek downstream of the impoundment, will continue to contribute tremolite fibers to Rainy Creek, regardless of the routing alternative selected.

Environmental Impacts: Environmental disturbance would be significant for a full diversion flood routing system, primarily from the massive excavations required to construct the diversion channels and drop chute. Environmental disturbance would be less for the east side channel than the west channel, but still significant. Channel lining with an impermeable material is recommended to prevent the complete loss of the smaller summer flows, and reduce potential for dam saturation. In order to construct an engineered channel that would have a reasonable longevity and acceptable maintenance, a large portion of either abutment would be removed, which creates an additional problem, namely, where to spoil the waste.

Additional concerns include relocation of the Forest Service access road at several locations, and the continued downstream flooding and erosion from the full 0.5 PMF flows.

Tailings/Dam Saturation: Saturation of the tailings dam and subsequent seepage and instability in the event of toe drain failure has been identified as a major regulatory concern. This subject has been addressed in detail by a study completed by Harding Lawson Associates (HLA) of San Francisco, California (Vahdani, 1992).

HLA completed a drilling program in the tailings and in the dam foundation materials as part of a study to assess the stability of the dam and impounded tailings during static and seismic loading conditions. The study concludes that the dam is currently safe under seismic load, even with the water at the face of the dam, and will not fail. The study encountered two types of tailings materials which appear to be interbedded and sloping away from the dam face. Elastic silts comprise about 60 percent of the tailings while loose, poorly graded sands and silty sands comprise about 40 percent. The elastic silts were not expected to liquify in a seismic event; however the sands could liquify if they remain saturated. If a section of the dam were to be removed the tailings could be expected to fail, but would maintain a 4:1 angle of repose. HLA judges the potential for material run-off in the event of a failure to be very low on the basis of its findings.

The drilling also indicated that the tailings consolidated with depth and gained significant strength. If the tailings are left without standing surface water, up to 5 feet of surface subsidence is projected in areas of deeper tailings as excess pore water pressure is relieved. HLA sees the major threat to dam stability to be the eventual failure of the toe drain piping. It will then be possible for the phreatic surface to increase in the dam and possibly begin seeping from the dam face. Should this occur there will be the likelihood of erosion of the toe and eventual weakening of the dam. Installation of additional piezometers is recommended to provide better monitoring and a conceptual design for a permanent drain structure to be retrofitted as required is proposed. HLA has indicated that based on the probable hydraulic conductivity of the tailings material, it may be possible to reduce the phreatic surface in the dam permanently by maintaining the pond surface approximately 500 feet upstream from the crest of the dam.

Diverting flows around the tailings impoundment will not eliminate saturation of a portion of the tailings dam adjacent to the channel, unless an impermeable liner is installed. The material covering the bedrock on the abutments is glacial outwash and till with moderate to very high permeability (Lewis, 1971). Significant loss of water through infiltration would be expected. The area of influence from the lost water is unknown but is likely to impact a portion of the tailings dam.

Infiltration could be eliminated by lining the channel with an impervious liner material, possibly HDPE or clay. Depending on the life of the selected material, infiltration would be significantly reduced or eliminated, at least through the life of the liner. Channel lining is an option with each alternative, hence no advantage or disadvantage to a particular alternative.

__Sedimentation:__ Reduction of downstream sedimentation associated with the tailings would be expected with a full diversion, particularly if the second (full diversion) option were exercised. The surface of the impoundment is currently devoid of vegetation and subject to potential erosion in major storm events despite its relatively low angle of repose because of the small particle size of the fine tailings. Over the next three to five years, it is anticipated that vegetation will become firmly established on the both the fine and coarse tailings and the potential for erosion will be greatly decreased. Sediment contribution from the tailings should become relatively insignificant.

Disadvantages associated with diversion include the loss of settling and natural filtration associated with some of the other options which provide a wetland in the upper portion of the tailings impoundment. While the impact of sedimentation from tailings materials may be lessened, there is a great potential for increased sedimentation from other sources associated with the massive excavations which would be required for the channels, drop chute, and other diversion structures.

In summary the full diversion alternates greatly reduce safety, increase the possibility of system failure, increase environmental disturbance, and increase construction and maintenance costs. Concerns over geotechnical stability, asbestiform fibers, and tailings dam saturation are not eliminated.

Advantages (Table 4.2) and disadvantages (Table 4.3) of the full diversion alternates are summarized below:

Table 4.2. Advantages associated with a full diversion flood routing system.

ALTERNATE	FULL DIVERSION - ADVANTAGES
All Alternates	• __Possible__ reduction in downstream tremolite fiber concentration in surface water; • Probable reduction in short-term sedimentation from the tailings impoundment.
Common Diversion Dam (Alternate Ia)	• Provides the least complex design for intercepting flows from both Rainy Creek and Fleetwood Creek.
Separate Diversion Dams (Alternate Ib)	• Intercepts water from both Rainy Creek and Fleetwood Creek before contact with any portion of the tailings impoundment area.
West Channel (Alternate Ic)	• Best overall alternate of full diversion channels; • Most stable geotechnically of the full diversion alternates.

ALTERNATE	FULL DIVERSION - ADVANTAGES
East channel (Alternate Id)	• Less environmental disturbance than west channel; • Bedrock channel reduces infiltration and subsequent potential for saturation of tailings dam.
Pipeline (Alternate Ie)	• Eliminates infiltration and subsequent saturation of tailings dam; • Least (long-term) environmental disturbance of diversion alternates; • Least public accessibility to flood flows, excluding inlet; Eliminates need for separate drop chute.

Table 4.3. Disadvantages associated with full diversion flood routing system.

ALTERNATE	FULL DIVERSION - DISADVANTAGES
All Alternates	• Does not use the reservoir capacity to temporarily store peak flows resulting in higher peak flows downstream in Rainy Creek; • Will not eliminate tremolite fiber contamination of downstream surface water; • Construction of diversion dams in tailings is very difficult, and the long-term stability of such dams is questionable; • Significant environmental disturbance to construct channels/pipeline to carry diverted flows around the tailings impoundment. Massive cut slopes would be required; • Diversion dam(s), channels, and other structures will be required to handle 0.5 PMF flows, making them large and very costly; • Dam safety is inferior. Dams in series are more prone to catastrophic failure; • No backup flood routing system; • The diversion channels will not eliminate the possibility of tailings dam saturation and resultant stability concerns, unless impermeable lining is installed; • Does not take advantage of the wetland within the tailings facility for settling and natural surface water filtration; • Increased maintenance; • Tailings will be dry, thereby increasing the possibility of blowing dust and raising air quality risks; • Limited opportunity for wetland habitat construction.

ALTERNATE	FULL DIVERSION - DISADVANTAGES
Common Diversion Dam (Alternate Ia)	• Does not achieve complete isolation of streamflows from tailings materials.
Separate Diversion Dams (Alternate Ib)	• More complex design; • Greater environmental disturbance resulting from construction.
West channel (Alternate Ic)	• Significant environmental impact from the massive excavations required to properly construct a long-term channel; • Channels are prone to plugging with debris, particularly during flood events, resulting in greatly increased risk of channel/system failure and associated safety risks; • Channel would require lining to prevent infiltration into underlying material, particularly during low flows; • Major relocation of the Forest Service access road would be required.
East channel (Alternate Id)	• Upper reach of channel in geotechnically unstable coarse tailings material; • Lower portion partially within fine tailings, or would require drilling and blasting of bedrock to construct channel; • Channel would be prone to plugging with associated safety risks; • Construction difficulties; • Channel lining would be required in coarse tailings section to prevent water loss and foundation problems; • Significant environmental impact, although less than west channel.
Pipeline (Alternate Ie)	• Very large (>20 ft diameter) pipe required to carry the full 0.5 PMF design flows; • Very expensive construction and material costs; • Geotechnical stability questionable; • Considerable maintenance required; • Prone to plugging, and once plugged, very difficult to clean; • Safety concern (no escape) from a closed system.

4.3 CHANNEL RECONSTRUCTION THROUGH THE IMPOUNDMENT

Initial studies of this concept were approached from the standpoint that a 0.5 PMF event could be safely routed through the Impoundment and discharged through the dam into a channel or drop structure constructed to withstand such a massive flood event. Large, armored channels similar to those required for a full diversion were the result. These concepts suffered from many of the same stability problems that were cited for the full diversion alteruatives. A study of the flood surges and the damping effect caused by the unfilled volume of the tailings impoundment suggested that the most useful feature of this concept is the potential for storing much of the runoff from events comparable to a PMF and releasing it downstream at a much reduced and more manageable volume. Our investigation centered on designs which would take advantage of this as it provided the safest method of passing a flood event equal to or exceeding a 0.5 PMF event, while adequately addressing the majority of the engineering, envfironmental and geotechnical concerns.

A general concept employed in these alteruates is to hold water away from the dam during all but veıy large runoff events. This principal of design results from the work of Harding Lawson Associates on geotechnical stability of the dam. The study showed that although the dam would not fail with water at the face even during an earthquake, additional stability and a reduced risk of foundation saturation could be obtained by keeping water back some distance thereby lowering the phreatic surface at the dam. We considered two concepts for providing this increased level of stability and several options for passing water through the dam face. These alteruatives are described in Section 4.3.1 below.

4.3.1 Description of Conceptual Designs

Water Level at 2904' (Alternate IIa): This alternate would allow inflows from Rainy and Fleetwood Creeks to enter the impoundment unimpeded. Once in the reservoir, the flows would be temporarily stored, or passed dfrectly through the impoundment with a constructed channel, depending on the volume received. This alteruate provides for a water elevation in the impoundment of 2904 feet which is the minimum practical elevation that can currentiy be obtained through control at the decant tower. Tailings materials have accumulated to this level at the decant tower.

Discharge froru the impoundment would be controlled at the tailings dam by a control structure, preferably a single concrete box culvert. The control structure would limit outflows to a maximum design flow (about 15 percent of 0.5 PMF). At this design rate the impoundment can receive a 0.5 PMF event without overtopping the dam.

An extensive study of outlet control structures was made before selecting the box culvert design. The control structure must necessarily have a small cross-sectional area if it is to reduce the volume of discharge and fully utilize the impoundment storage capacity. More natural control structures such as open channels were considered but these could only be utilized by sacrificing a large portion of the impoundment's potential storage capacity. Pipelines were also considered as an inexpensive alternative but these presented safety hazards and were judged to be more subject to failure in long-term service.

Outflows from the control stmcture would be returned to Rainy Creek by an engineered channel armored with a rock rip-rap lining, integrating a series of reinforced concrete drop stmctures. The channel would be considerably smaller than a full diversion channel, and would be designed to incorporate natural terrain where possible to promote aesthetics and decrease environmental disturbance. Plate 8 shows a plan view of this routing alternate.

An emergency spillway, designed to safely pass flows exceeding the 0.5 PMF without overtopping or causing damage to the dam, could be constructed with this system. The spillway would be located opposite the control stmcture and outflow channel to prevent interference during use. A conceptual plan of a spillway located at the west abutment is shown on the plan. The spillway would be constmcted such that flows are carried past the toe of tiie dam before release in order to prevent damage to the dam.

Water Level at 2910' (Alternate IIh): The fine tailings in the impoundment are saturated, unconsolidated, and have littie bearing capacity making standard construction methods and equipment difficult to use. Due to the expected difficulties associated with constructing the inflow channel in the fine tailings, a variation of this altemate was investigated. To reduce the problems of construction in the tailings materials, a low level dike of cohesive (low permeability) material would be constmcted across the tailings impoundment, approximately 500 feet from the face of the tailings dam as recommended in the Harding Lawson Associates dam stability report. Located at this distance from the dam the potential impact of standing water on dam foundations is minimal in the judgement of engineers at Harding Lawson Associates. Top of dike elevation would be approximately 2912.0, with the water level in the impoundment maintained at 2910.0, which has been selected as the maximum practical elevation at which water can be maintained in the impoundment without significant loss of storage capacity or increasing the risks associated with saturated tailings dam foundations and sudden failure or breaching of the dike. By raising the water level in the impoundment, the length of inflow channel and subsequent tailings excavation would be reduced and this would reduce construction costs. This altemate provides water cover for much of the tailings and thereby reduces the potential for dust production and also reduces the areal extent of required revegetation. Plate 9 shows a plan view of this altemate.

There are some additional risks with this alternative, however. Should the dike leak, which it may very well do because of the difficulty in getting good compaction of the dike materials on top of tailings and the potential for seepage through the taihngs material itself, a drainage channel would probably be needed below the dike. Also, in the event of a major runoff event, one slightiy greater than a 100-year storm, the dike would be overtopped resulting in damage to it and to the drainage channel below the dike.

East Abutment Outlet (Alternate IIc): Placing the control stmcture and outflow channel on the east abutment, anti the emergency overflow channel on the west abutment, as shown in Plate 8, is judged to be the best overall alternate for routing floods through the vermiculite tailings impoundment while maintaining stmctural integrity. Placing the outflow on the east abutment provides the most aesthetically pleasing alternate for returning the flows to Rainy Creek, with the least environmental disturbance of considered alternatives.

The east abutment area can easily be modified to constrnct the outflow chamel without significantiy disturbing the area. The outflow channel would be armored with a rock rip-rap lining and integrate a series of drop stmctures placed to take maximnm advantage of the terrain. A natural drainage would be incorporated into the final design to increase aesthetics, and decrease excavation and construction costs.

The emergency spillway, if provided, would be constructed ui natural material adjacent to the tailings dam on the west abutment to the eatcnt that it did not interfere with the existing Forest Service road. The area is presently disturbed from mining activities. To protect the toe of the dam, the spillway will carry the flows past the toe before release. The excavated material would be placed in the groin of the darn for additional protection.

The primary disadvantage of this alternate is the longer inflow channel in the tailings, resulting in higher construction costs to excavate and construct the channel. Some drilling and blasting may be required to constrnct portions of the outflow channel as well.

West Abutment Outlet (Alternate IId): Locating the outflow control stmcture and channel on the west abutment, and the emergency spillway on the east abutment was investigated as an alternate for returning flows to Rainy Creek downstream of the tailings dam. No plans are provided for this alternate.

The primary advantage of this alternate would be to shorten the inflow channel through the tailings, reducing the extent of specialized construction to bulld the channel. Because the tailings are not as deep on this side of the impoundment, both the length of the channel excavation and the quantity of material to be removed would be reduced.

The primaiy disadvantage is the steeper sideslopes making construction of the outflow channel more difficult, and with questionable long-term geotechnical stabillty. A concrete drop chute (at considerable cost) or significant excavation of the abutment area may be required. Placing the emergency spllhvay on the east abutment would reqnire relatively more excavation, partially in undisturbed forest, to get the flow past the toe of the darn before releasing it, reducing visual aesthetics as well. A partial relocation of the Forest Service access road would be required. Due to these engineering and aesthetic draw-backs, and lack of discernable advantages, this alternate was eliminated.

<u>Outlet Over Dam Face (Alternate IIe):</u> Constructing an outiet through the center of the dam and down the face was investigated as an alternate for returning flows to Rainy Creek. This alternate would consist of a straight inflow channel through the fine tailings connected to a reinforced concrete control structure and drop chute. Plate 10 provides a plan view of this alternative.

Placing an outiet in the dam face eliminates the need for excavation of either abutment, unless an emergency spillway is desired. The outiet control structure and drop chute would be built as one structure, and tied directiy into the existing channel below the dam, eliminating the need for extensive downstream work. Overall, environmental disturbance is negligible.

There is an increased possibility of tailings dam saturation and seepage with this option. The zone of influence from the channel will affect a larger area than if it were located adjacent to an abutment. As with the other alternates, lining the channel would eliminate the problem. Long-term geotechnical stability of this system may be questionable, and construction would be moderately difficult on the steep slope.

Other disadvantages are reduced aesthetics, higher construction costs (reinforced concrete) and public safety (straight-walled drop chute and high velocities eliminate any chance of escape).

4.3.2 Evaluation of Alternatives for Channel Reconstruction in the Tailings

<u>Safety:</u> Routing floods through the tailings impoundment provides the best method to <u>safely</u> pass storm events of 0.5 PMF or larger while assuring the integrity of the dam. This concept takes advantage of the temporary storage capacity of the impoundment to reduce outflows while providing safe, effective flood routing.

The existing tailings dam is geotechnically very stable, having been designed to withstand earthquakes of a recommended magnitude with no loss of integrity. Temporarily storing peak flows provides a way of assuring minimum risk to the dam. Elimination of upstream diversion dams associated with the other main alternatives reduces risks associated with diversion dam failure.

Because of the storage capacity in the reservoir, and the emergency spillway, risk from debris/plugging is minimal for this alternative. In addition, several low maintenance structures would be installed to prevent debris from entering the control structure. During peak events, the entrance into the control structure will be submerged to prevent debris from entering into the control structure.

Reduced peak outflows will result in a considerably smaller outflow channel, making escape from the channel easier, hence better for public safety. In addition, the reduced outflows result in less flood damage to downstream structures, such as the highway.

Water Quality Impacts: With this alternate, tremolite fibers from the coarse tailings dump, and fine tailings impoundment will continue to impact surface water. However, during normal flow conditions the low gradient of the reconstructed channel and the placement of protective cover in the reconstructed channel will greatly reduce the risk of tremolite entrainment. Also it is anticipated that entrainment will continually decrease as vegetation becomes established and stabilizes the dump, impoundment, and other disturbed areas. Preliminary data from water monitoring programs indicate that water quality degradation from other mineral constituents is minimal at this site.

As discussed in Section 4.2, tremolite fibers will not be eliminated from Rainy Creek, regardless of the alternative selected. Fibers from the headwaters of Fleetwood Creek, from Carney Creek, and in the Rainy Creek streambed downstream of the impoundment, will continue to contribute to fiber counts in Rainy Creek.

Environmental Impacts: Environmental disturbance will be minimized with this alternate, especially when compared with full diversion. Some disturbance will occur during construction of the outflow channel. By reducing outflow volumes, erosion and other flood-related problems will be diminished.

Tailings/Dam Saturation: Saturation of the tailings dam in the immediate vicinity of the inflow channel, and resulting embankment stability should the toe-drains become inoperable, is a primary regulatory concern. Because of the low permeability of the fine tailings relative to the dam material, major water loss through infiltration is not expected to be as severe of a problem as with the diversion channels. Further, the rate of water movement through the fine tailings is significantly slower than the dam, as demonstrated by the piezometers installed in the dam face. Water entering the dam from the tailings or channel is expected to drain relatively quickly, hence reducing the possibility of saturation and subsequent seepage.

As discussed earlier, diverting flows to the side of the impoundment will not eliminate the possibility of tailings dam saturation. The only sure method of eliminating the risk, from any alternate, is with an impermeable channel or pipeline. Should tailings dam saturation become a problem, construction of an engineered toe drain will be completed by W.R. Grace.

Sedimentation: Increased sedimentation from the tailings impoundment is expected for a short period of time (estimated at 2 to 5 years) following closure. After that, vegetation will become established and provide slope stabilization, reduced erosion, utilization of excess water, and wildlife forage. A detailed description of re-vegetation is provided in Section 5.7 Sedimentation associated with channel excavations and other construction activities may also occur for a short time period, but will be negligible compared with a full diversion alternate.

Routing the surface water flows through the impoundment will take advantage of the remaining wetland to improve water quality through natural filtration and settlement.

In summary, routing floods through the existing tailings impoundment with a controlled outflow system provides the best method to safely control flood events meeting or exceeding the required 0.5 PMF design. This general concept provides a feasible method to safely route floods while minimizing environmental disturbance and maintenance, and improving aesthetics.

Advantages (Table 4.4) and disadvantages (Table 4.5) of routing the flood flows in a reconstructed channel through the tailings impoundment follow:

Table 4.4. Advantages associated with routing floods through the tailings impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - ADVANTAGES
All Alternatives	• Provides a higher level of public safety than other alternatives while retaining a relatively simple design; • Provides a safe, cost effective method to handle storm flows while maintaining long-term integrity of the dam; • Geotechnically the most stable alternative; • Plugging/debris problems less critical or likely; • The system is capable of handling floods larger than 0.5 PMF with the addition of a relatively simple emergency spillway; • Outflow channel relatively small, making construction feasible and cost effective; • Limited environmental disturbance; • More natural/aesthetic outflow channel; • Remaining wetland provides improves surface water quality through natural filtration and settling; • Water loss to infiltration expected to be minimal; • Less overall maintenance; • Reduced potential for airborne particulate; • Reduced outflows will reduce downstream impact from flooding.
Water Level at 2904' (Alternate IIa)	• Maintains water away from the dam face as much as possible for maximum safety.
Water Level at 2910' (Alternate IIb)	• Maintains water away from the dam face provided seepage through or under the dike is minimal; • Reduces the requirements for construction in mucky material; • Reduces requirements for revegetation; • Maximum potential for reduction of airborne particulate from the impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - ADVANTAGES
East abutment outflow (Alternate IIc)	• Less overall environmental disturbance than west side outflow channel; • Existing terrain can be easily modified for outflow channel thereby reducing environmental disturbance; • Emergency spillway on west abutment can be constructed with a minimum of excavation and disturbance; • Highest public safety of all alternates.
West abutment outflow (Alternate IId)	• Shorter inflow channel; • Bedrock does not affect construction.
Outflow over dam face (Alternate IIe)	• Eliminates excavation of abutments for outflow channels; • Negligible environmental disturbance; • Control structure and drop structure are one structure; • Minimal downstream work required.

Table 4.5. Disadvantages associated with routing floods through the tailings impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - DISADVANTAGES
All Alternates	• Inflow channel difficult to construct in fine tailings, requiring specialized construction methods and equipment and increased costs; • Does not address tremolite fiber issue actively; • <u>Possible</u> saturation of a portion of the tailings dam; • Probable increased short-term sedimentation; • Slight risk of control structure becoming plugged.
Water Level at 2904' (Alternate IIa)	• Potentially difficult construction of a long channel through soft mucky tailings.
Water Level at 2910' (Alternate IIb)	• Dike and foundation materials may seep at a significant rate creating saturated tailings downstream of the dike, thereby defeating its intended purpose; • A major runoff event will cause the dike to be breached and repair will be required; • Reduces slightly the total storage capacity of the impoundment.

ALTERNATE	ROUTING THROUGH IMPOUNDMENT - DISADVANTAGES
East abutment outlet (Alternate IIc)	• Excavation of bedrock may be necessary to construct outflow channel; • Longer inflow channel required, unless variation is selected.
West abutment outlet (Alternate IId)	• Outflow channel difficult to construct on steep side slopes; • May require concrete drop chute; • Emergency spillway difficult to construct on east abutment; • Portion of the Forest Service access road requires relocation or reconstruction; • Increase environmental disturbance.
Outlet over dam face (Alternate IIe)	• Long-term geotechnical stability may be questionable; • Saturation of tailings dam more likely than with other (no diversion) alternatives; • Concrete structures increase cost; • Safety concern with vertical side walls and high velocity flows; • Most unnatural of impoundment routing alternatives.

4.4 PARTIAL DIVERSION

A partial diversion of flood flows would entail diversion dam(s) and channels designed to intercept and divert flows up to and including a selected design flow, i.e. 10-year or 100-year events, which are described in Section 4.4.1 below. Flows exceeding the design capacity of the diversion dams would be allowed to by-pass the diversion dam through a "blow-out" plug of uncompacted fill placed in an engineered spillway and be routed through the reservoir using a system similar to those in Section 4.3. The concept behind this alternative would be to provide a system that would combine the advantages of a full diversion system with the advantages of flood routing through the reservoir. A full engineering analysis of these alternates is not detailed below, as many of the issues are covered in previous sections.

4.4.1 Description of Conceptual Designs

<u>**100-Year Flood Diversion (Alternate IIIa):**</u> A partial diversion system would require one or more dams similar to the full diversion dams, but designed to allow higher flows to by-pass them during larger events. The smaller design flows would be diverted around the impoundment in an open channel or pipeline, returning to Rainy Creek below the tailings dam. The larger flows would be routed through the reservoir using a system similar to those in Section 4.3, which includes a constructed

inflow channel and outflow channel, control structure, and emergency spillway. This alternative would virtually "double" the costs for tlie project, by requiring both flood routing systems to be constmcted.

Designing and constructing a stmcturally competent diversion dam capable of diverting smaller flows whlle by-passing larger flows will be difficult to accomplish. As stated earlier, the taillngs do not provide adequate foundation for stractures, making long-term structural integrity and durabillty questionable. A single by-pass flood event would llkely cause irreparable damage to the diversion stracture due to scouring of the foundation layer. Constracting separate dams for Fleetwood and Rainy Creeks above the tallings is again an option. Regardless of the diversion dam site selection, continual maintenance would be required.

Due to the adverse conditions associated with the east side (coarse tallings, bedrock, etc.), the partial diversion channel would be constmcted on the west side of the iinpoundruent. Assuming a 10 ft. flat-bottomed channel, 2:1 maximum cut slopes, and 0.005 ft/ft gradient produces the chamel section shown in Plate 11. The bottom of channel elevation would be approxiinately 2888 at the tailings dam. As with a fnll diversion channel, masshe cuts would be required to construct a channel that would provide long-term service. Complete relocation of the Forest Service access road would again be required.

During a 0.5 PMF event, assuming the impoundment routing system was constracted sinilar to those in Section 4.3, the water level in the impoundment would rise to at least 2922, making the water level in the partial diversion channel 34 feet in depth (refer to Plate 11). Obviously, this volume of flow would be impossible to control without a structure, further increasing the cost of this system whlle providing llmited added benefit. Lining the channel would also be recomruended to prevent infiltration, geotechnical instabllity, and possible taillngs dam saturation.

An option would be to install a pipeline to carry the partial flows around the impoundruent, ruaking the system sinilar to the existing Rainy Creek diversion pipeline. Continual maintenance could be expected based on W.R. Grace's experience with the current pipellne, and plugging would be a problem. A pipellne system of any kind is not recommended.

A partial diversion system would require separate outflow chamels for the diversion channel, and the "backup" impoundment routing system. The outflow channel for the impoundment would be constructed as described in Section 4.3, whlle the partial diversion would require a drop chute or some other method of returning outflows to the elevation of Rainy Creek downstream of the tallings dam.

<u>10-Year Flood Diverslon:</u> The partial dhersion of stream flows exceeding a 10-year stonn event would be virtually identical to the 100-year event. The restrictions of constraction equipruent dictate that the diversion chamel would assume basically the same dimensions. The only significant design variation is in the outiet control structure irom the diversion dam(s) which needs to be more restrictive in order to

limit flow. The smaller outiet, is a potential source of problems in that will be more subject to plugging by debris and wlll likely require more frequent cleanout.

One perceived advantage of this alternate is the periodic wetting of the tailings which might be beneficial for maintenance of vegetation and reduction of potential dust production. However, this wetting would be incomplete at best and its benefits would be questionable on such an infrequent and unpredictable basis.

4.4.2 Evaluation of Partial Dhrersion Alternatives

Safetv: From a safety standpoint, partial diversion does not improve safety over the "no dlversion" alternate, however, it is significantiy better than a full diversion system. The reasons are covered in previous sections. Plugging or fallure of smaller partial diversion dams would be less critical.

Water Quality Impacts: Asbestiform fiber contamination of surface water from the tailings impoundment would be reduced by diverting the "day-to-day" smaller flows around the unpoundment, but would not be ellminated as discussed in Section 4.2.

Envtronmental Impacts: The environmental disturbance would be the most significant of any option. Massive excavations would be requfred for the diversion channel and drop chute. All excavation required for the outflow channel associated with routing through the impoundment would remain as well. Downstream impact would be reduced when compared to full diversion, but would be greater than the alternates routing floods through the impoundment.

Tailings/Dam Saturation: The possibillty of saturating a portion of the tallings dam due to continuous flow through the impoundment will be eliminated, however, saturation from the diversion channel remains a possibillty unless channel llning is installed.

Sediment: Short-term sedimentation from the tailings impoundment would be reduced with this alternative, but may increase from the major excavations associated with the diversion channel. The advantage of using the impoundment wetiand for improving surface water quallty through natural filtration and settlng would be eliminated.

in summary, using a partial diversion system in conjunction with an impoundment routing system does not increase safety over the impoundment routing system. This alternate greatiy increases costs. Maintenance and environmental disturbance increase, and geotechnical stabillty, constraction feasibillty, tailings dam saturation, and sedimentation remain as issues.

Advantages (Table 4.6) and disadvantages (Table 4.7) of the partial diversion alternate are summarized below:

Table 4.6 Advantages associated with partial diversion flood routing systems.

ALTERNATE	PARTIAL DIVERSION - ADVANTAGES
All Alternates	• Will provides a higher level of public safety than a full diversion; • Geotechnically more stable than full diversion; • Plugging of channel from flood debris less critical than full diversion; • Possible reduction in downstream tremolite fiber concentration in surface water; • Possible reduction in short-term sedimentation from the tailings impoundment.
100-Year Flood Design Basis (Alternate IIIa)	• Diversion dam outlet structures will be less prone to plugging than those for a 10-year flood.
10-Year Flood Design Basis (Alternate IIIb)	• Periodic wetting of tailings <u>may</u> enhance growth of vegetation and provide for some degree of dust control; • Marginally lower costs for channel lining materials.

Table 4.7 Disadvantages associated with partial diversion flood routing systems.

ALTERNATE	PARTIAL DIVERSION - DISADVANTAGES
All Alternates	• Adds no safety benefit to impoundment routing (no diversion) alternative; • Partial diversion dams difficult to construct in fine tailings, requiring specialized construction methods and equipment and increased costs; • Long-term stability and integrity of partial diversion dams questionable; • Increases overall cost of the project significantly due to combination of systems; • Increased maintenance, particularly with partial diversion dams; • Saturation of tailings dam remains a possibility without diversion channel lining; • Possible increased short-term sedimentation from excavation; • Does not take advantage of impoundment wetland; • Plugging of smaller partial diversion channels; • Largest environmental disturbance of all alternatives.

ALTERNATE	PARTIAL DIVERSION - DISADVANTAGES
100-Year Flood Design Basis (Alternate IIIa)	• Tailings will not receive a thorough wetting on any reasonably short time frame.
10-Year Flood Design Basis (Alternate IIIb)	• More prone to plugging than a system designed for larger flows.

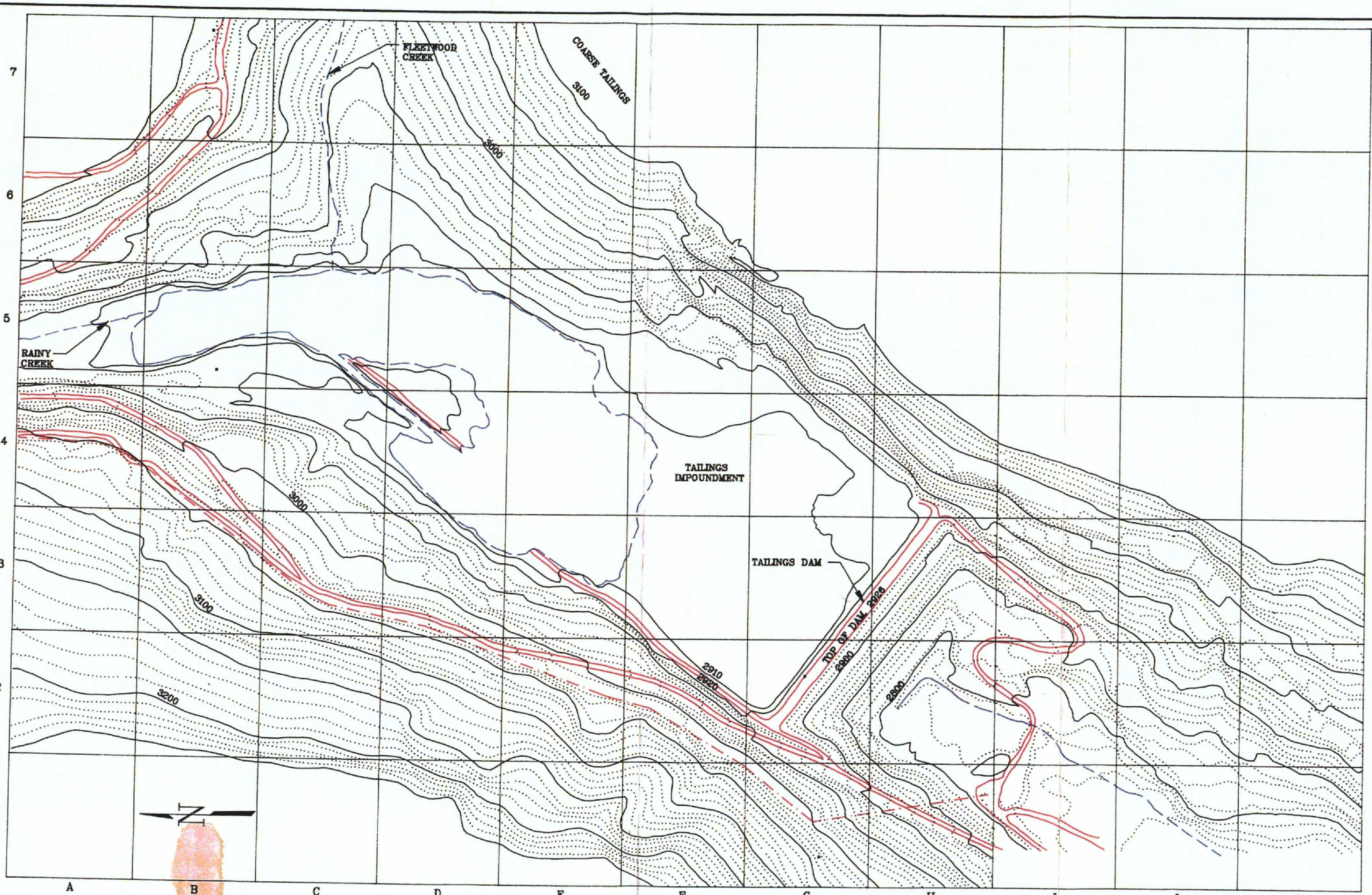
4.4 SUMMARY/CONCLUSIONS

Based on findings of the engineering analysis of the various flood routing alternatives, routing the flood through the tailings impoundment using a designed structure to control discharges to an east abutment outflow channel appears to be the best, most feasible, and safest method for flood routing Rainy Creek through the vermiculite tailings impoundment area. In our judgement, safety should be the overriding factor in selection of a permanent reclamation plan. This alternate provides sufficient storage capacity within the impoundment to receive a 0.5 PMF event without utilizing an emergency spillway which is provided in the event of an even larger storm.

The recommended alternative does violate a provision of the permit requiring diversion of water around mine wastes at closure. This could be a matter of concern from the standpoint of water quality issues. It should be understood that this mine is not a base metal mine and does not produce acid mine drainage typically containing high levels of metals. One significant area of potential concern is tremolite fiber entrainment. However, Rainy Creek is not utilized directly as a drinking water source. Other alternatives will not totally eliminate this concern since Fleetwood Creek and Carney Creek originate in areas where natural outcropping of tremolite occurs or which have been subject to disturbance by mining activity.

In order to reduce these concerns, disturbed areas will be stabilized to reduce erosion through the establishment of vegetative cover. Similar measures are proposed for the tailings impoundment to reduce the level of suspended particulate in surface waters discharged through the dam. Included in these measures will be revegetation of tailings beach areas and installation of channel linings to stabilize the channel and prevent direct contact with underlying tailings material. A program to establish current water quality levels is underway and will continue on a regular basis as reclamation proceeds. Overall entrainment of asbestiform fibers from the tailings should be minimal due to these design measures.

Another area of concern may be the establishment of a precedent for reclamation by allowing surface waters to be routed through mine waste facilities. Had this facility presented a significant risk to water quality, our recommendations would have been entirely different. As it is, the resolution of the safety and long-term stability aspects of the existing situation appear to take precedence over the relatively minor water quality issues, which are not life threatening or environmentally damaging. In summary, site-specific considerations make channel re-establishment a sound decision where at other facilities diversion may be more technically sound.

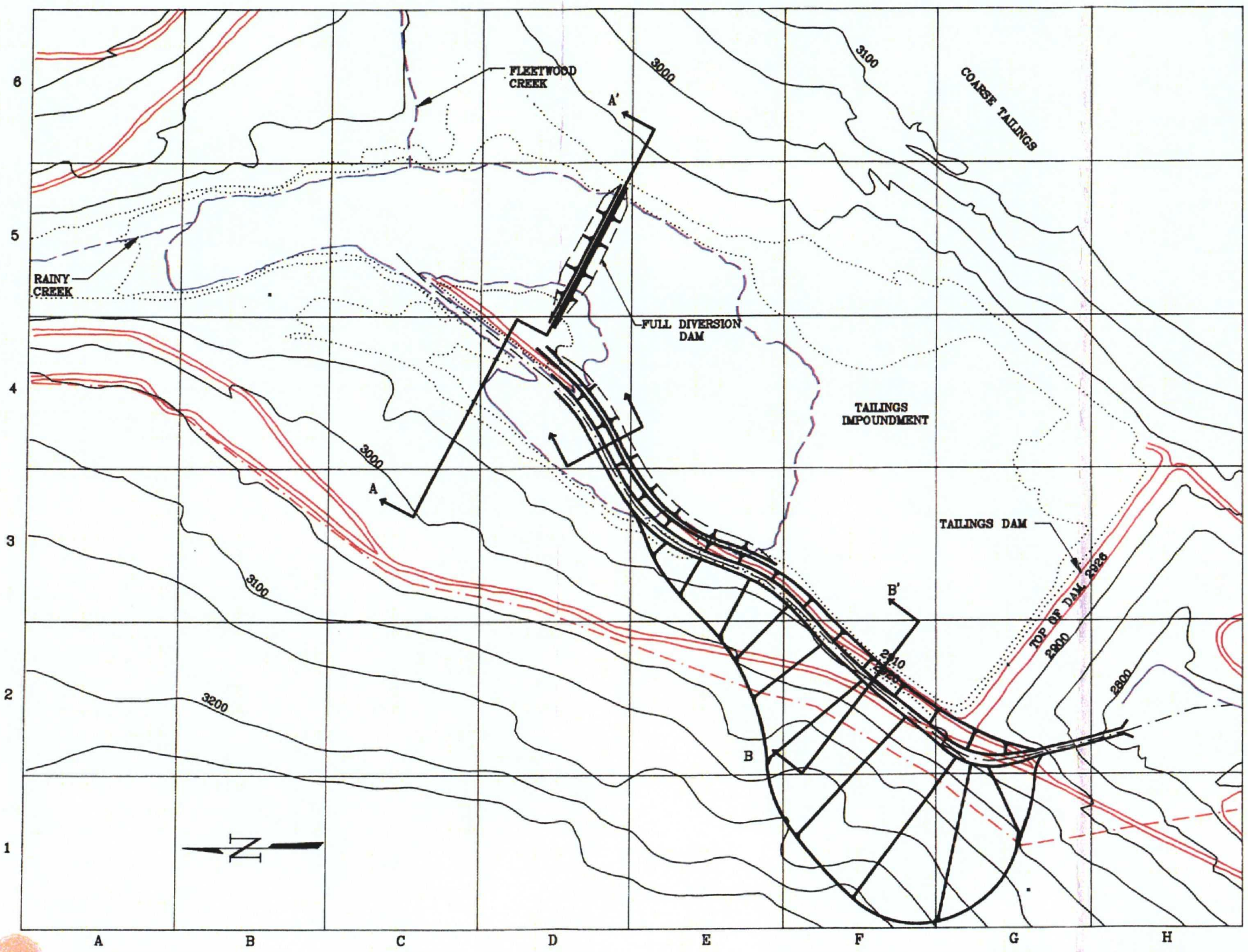


LEGEND

PIPELINE
WATER
ROAD
50' CONTOUR LINE
10' CONTOUR LINE
SCALE
0 100 200 300 400 500 600 700 800 900 1000

REFERENCE	REVISION

Plate 1. Plan view of project area



LEGEND

PIPELINE — 50' CONTOUR LINE

WATER — 10' CONTOUR LINE

ROAD — SCALE 0 100 200 300 400 500 600 700 800 900 1000

Plate 2. **Plan view of conceptual full diversion flood routing system employing a single diversion dam below the confluence of Rainy Creek and Fleetwood Creek. Cross-section typical of diversion dam construction indicated by Section A-A'.**

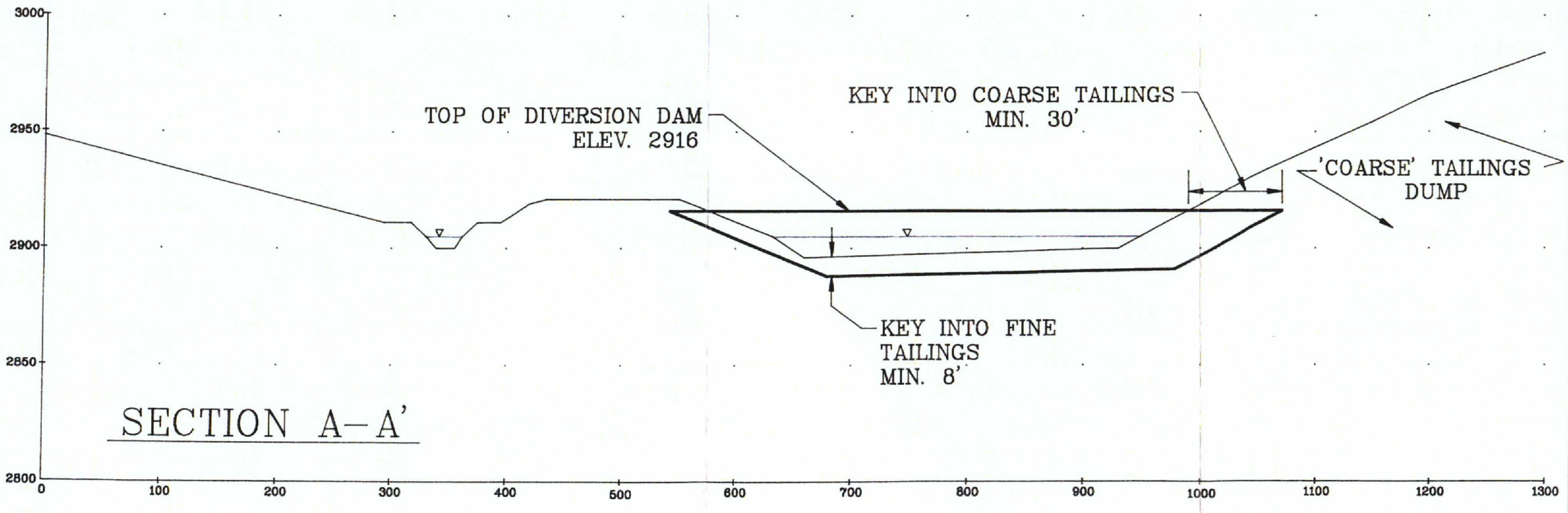
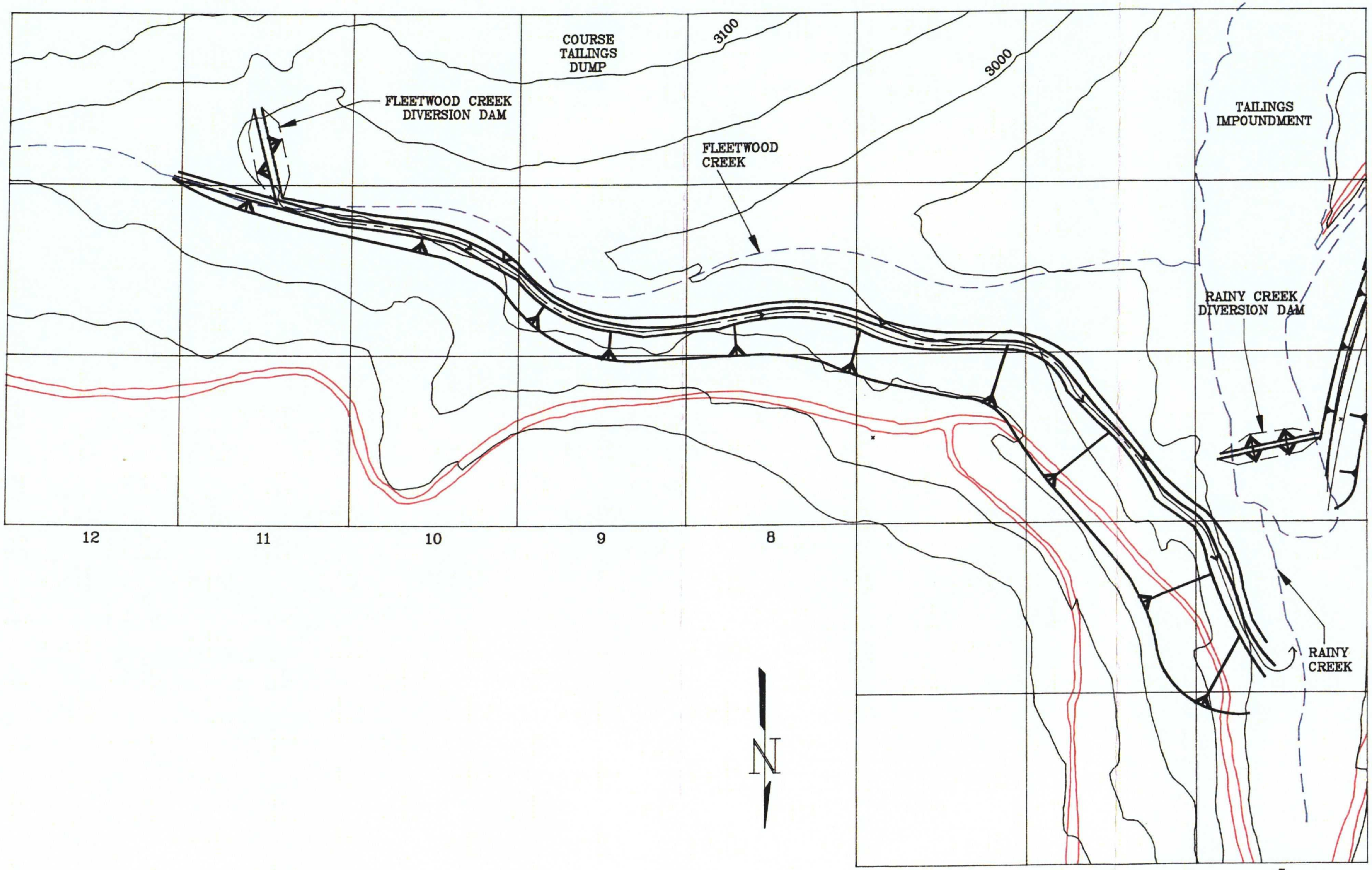


Plate 3. **Cross-section A-A' showing typical section of diversion dam for a full diversion flood routing system.**



LEGEND

PIPELINE
WATER
ROAD
50' CONTOUR LINE
SCALE
0 100 200 300 400 500 600 700 800 900 1000

Plate 4-A. Plan view of full diversion dam and channel to deliver Fleetwood Creek to the Rainy Creek diversion dam. Diversion dam located above coarse tailings dump. This figure to be matched with Plate 4-B.

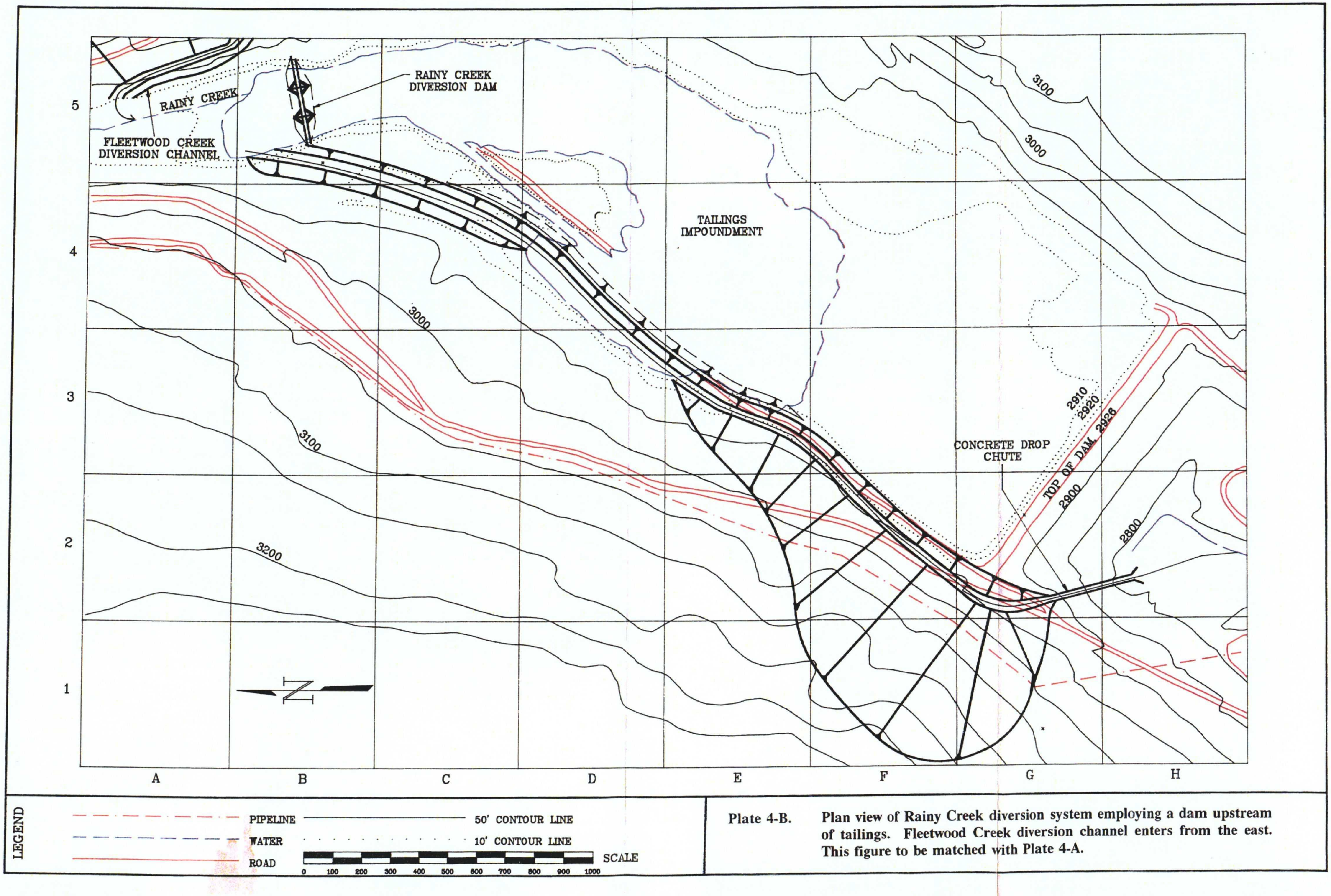


Plate 4-B. Plan view of Rainy Creek diversion system employing a dam upstream of tailings. Fleetwood Creek diversion channel enters from the east. This figure to be matched with Plate 4-A.

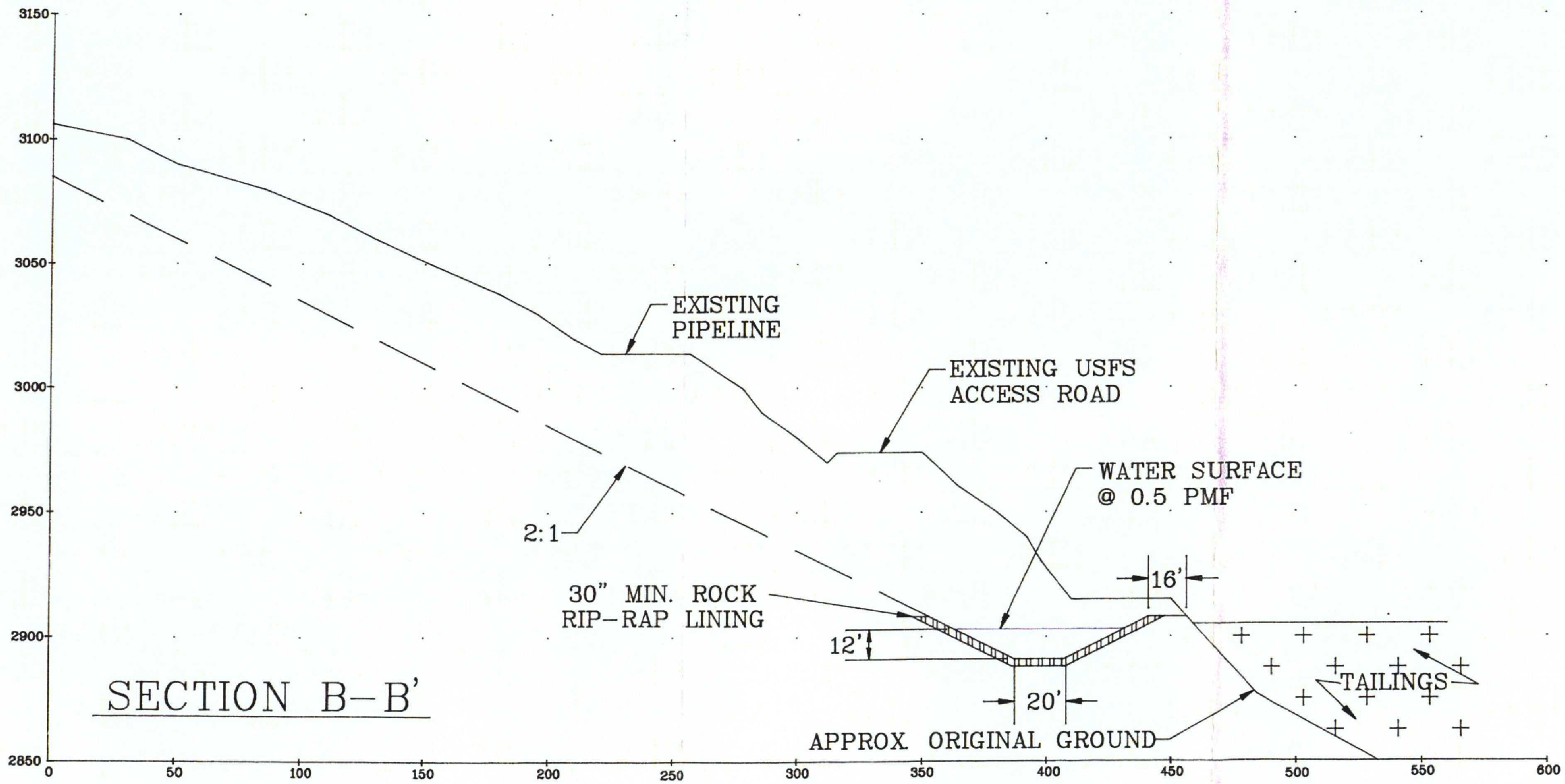


Plate 5. **Typical cross-section of west diversion channel showing limits of excavation.**

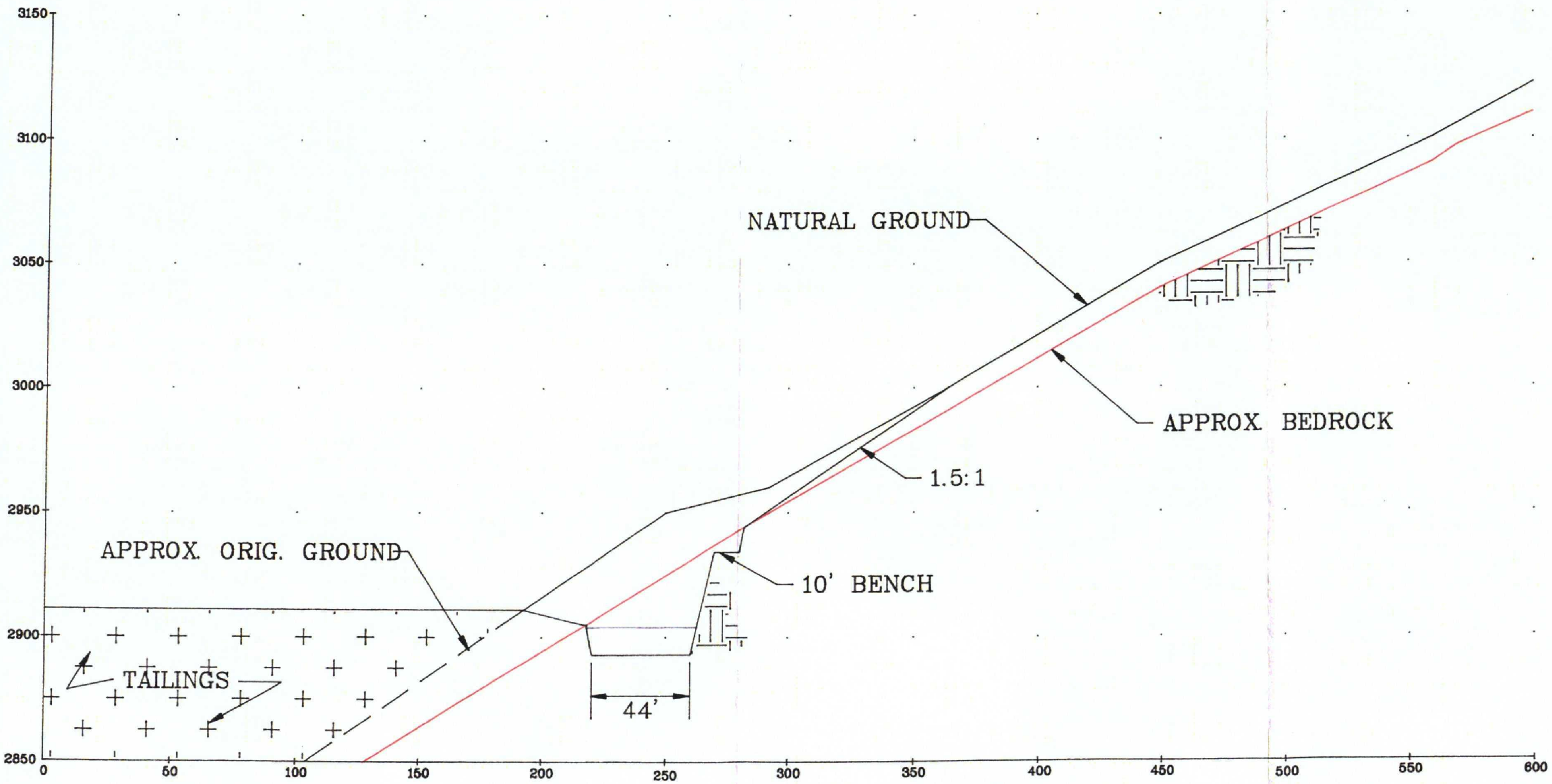


Plate 7. Typical section of the east side full diversion channel constructed in bedrock, outside of the fine tailings.

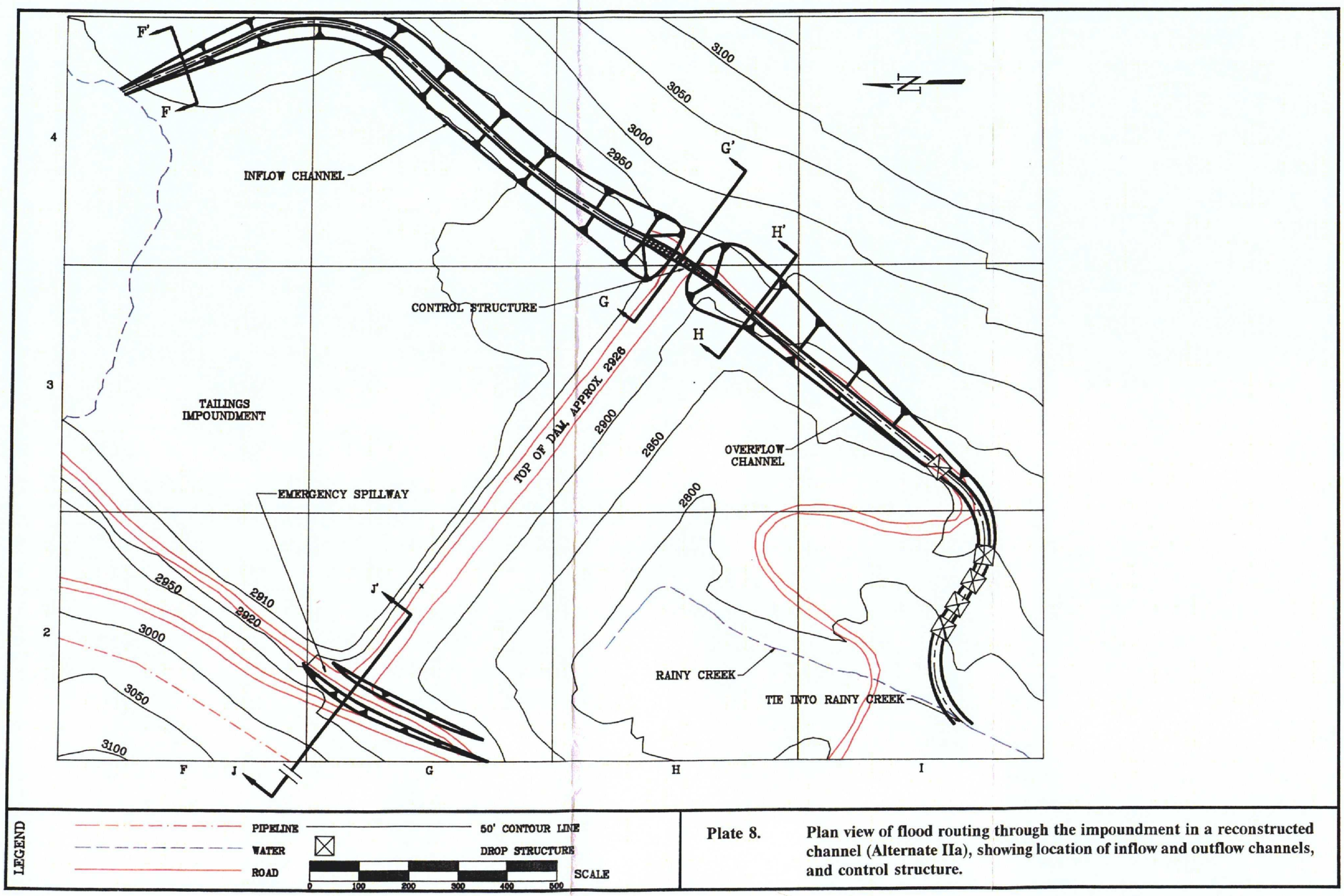
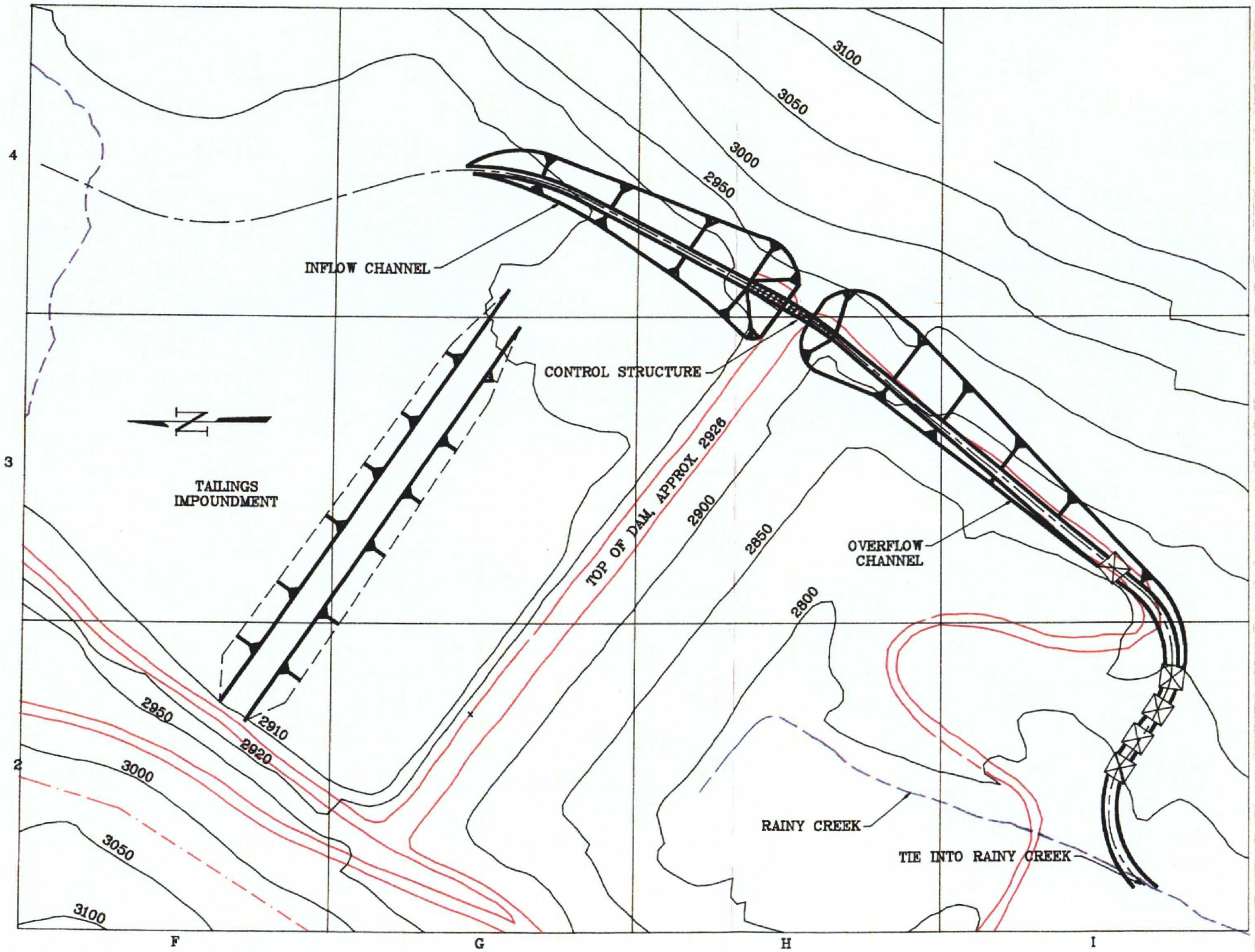


Plate 8. Plan view of flood routing through the impoundment in a reconstructed channel (Alternate IIa), showing location of inflow and outflow channels, and control structure.



LEGEND

PIPELINE — 50' CONTOUR LINE

WATER — DROP STRUCTURE

ROAD — SCALE 0 100 200 300 400 500

Plate 9. **Plan view of flood routing system through the impoundment with low profile dike (Alternate IIb) showing the dike and revised inflow channel. Other structures are the same as Plate 8.**

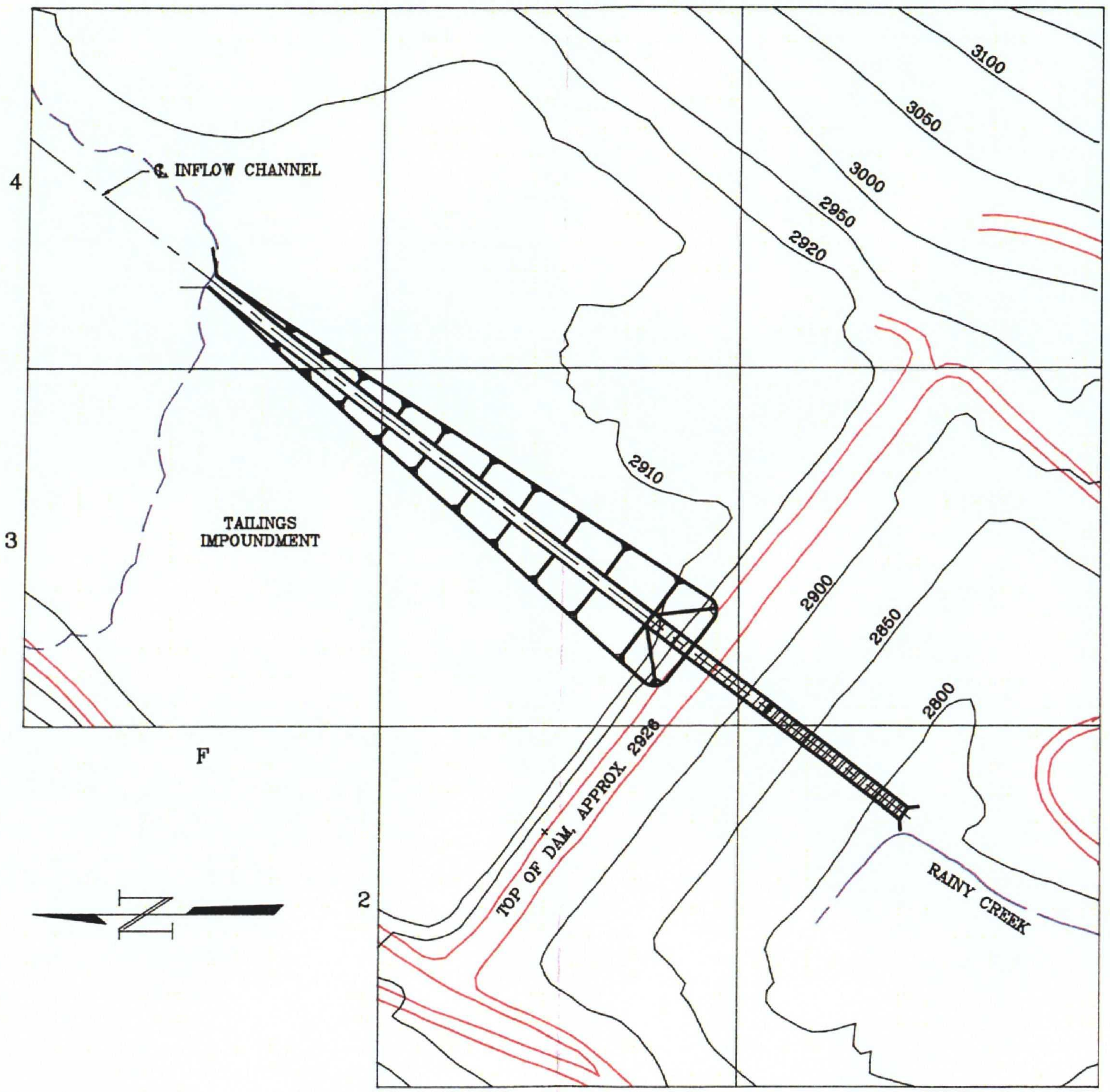


Plate 10. Plan view of outlet/control structure over dam face for routing floods through the tailings impoundment (Alternate IIe).

5.0 PROJECT DESIGN - PREFERRED ALTERNATIVE

5.1 GENERAL DESIGN APPROACH

The general design approach of the preferred alternative entails routing all flood flows from both Rainy Creek and Fleetwood Creek into the tailings impoundment, controlling discharge with a control structure, and returning water to Rainy Creek (downstream of the tailings dam) by means of an outflow channel. No diversion structures will be employed. Flows in excess of 0.5 PMF will be handled with an armored emergency spillway. Routing flows through the impoundment provides the safest method of passing major flood events through the impoundment area, while maintaining the long-term integrity of the tailings dam. The advantages of such a system have been demonstrated in Section 4.0.

Flood flows enter the impoundment unrestricted and, depending on the discharge rate, are passed directly through the impoundment and discharged, or temporarily stored in the existing reservoir until discharged. Discharges from the impoundment are restricted to a design peak outflow by means of a concrete box culvert. Discharges from the control structure enter a constructed outflow consisting of a rock-lined, trapezoidal channel connecting a series of concrete drop structures. Flows are returned to Rainy Creek approximately 800 feet below the tailings dam. An inflow channel will be constructed in the tailings in order to connect the impoundment wetland with the control structure. This system will allow W.R. Grace to maintain a relatively constant water surface elevation (in the wetland) to aid in revegetation, and prevent saturation of the tailings dam.

Other work proposed during closure includes removal of the existing water control structures (Rainy Creek diversion, emergency spillway, and decant tower); providing a stabilized Fleetwood Creek channel through the coarse tailings dump; revegetation and other erosion control and surface stabilization measures; and, general reclamation efforts to improve natural aesthetics of the impoundment area.

Plate 8 shows a plan view of the tailings impoundment with the preferred flood routing alternate overlain. Following sections provide greater detail of the proposed closure plan for the vermiculite tailings impoundment.

5.2 TAILINGS IMPOUNDMENT

The tailings impoundment will basically remain as it currently exists with a pond and associated fringe of emergent vegetation (wetland), "beach" area, dam, and inflow from Rainy and Fleetwood Creeks. The flood routing system will be constructed, the existing water control structures removed, and revegetation/reclamation work completed during closure.

Following closure, the pond will be retained as a natural wetland. The wetland will have a water surface elevation of 2904 $\pm$ and will encompass approximately 20 acres in the middle to upper portion of the impoundment. Water depths will range from 0 ft at the water's edge to a maximum of about 7 ft, with an average depth estimated at 2 to 4 feet. The water's edge will remain approximately 700 to 800 feet from the dam creating a "beach" area (between the water's edge and the upstream face of the dam) of slightly less than 20 acres. Revegetation will take place on the entire impoundment area (see Section 5.7). The estimated boundaries of the wetland, following closure, are represented by blue lines on Plate 8.

Inflow from Rainy Creek will continue to enter the impoundment from the north. The (Rainy Creek) diversion structure, located approximately 1 mile upstream of the impoundment, and associated pipeline together with the present emergency spillway and decant tower/pipeline will be removed. Fleetwood Creek will be restored to a stabilized channel located adjacent to the toe of the coarse tailings dump, and enter the impoundment from the east. Neither flow will be restricted or diverted.

A flood routing control system for the impoundment will be constructed on the lower (dam) end. Details are located in following sections.

5.3 INLET CHANNEL

An inlet or inflow channel, from the edge of the wetland to the control structure, will be constructed as part of the preferred flood routing system. In addition to flood routing, the inflow channel will provide passage for low flows through the impoundment to prevent the water surface elevation in the pond from rising, inundating the beach area, and eventually saturating the tailings dam. The inlet channel is shown on Plate 8.

The inflow channel will connect the wetland with the control structure. The channel crest elevation (at the edge of the wetland) will be set at 2904.0 $\pm$, and the crest elevation of the control structure will be set at 2900.0, making a channel gradient of approximately 0.0038 ft/ft or 0.38%. Maximum calculated flow velocity in the inflow channel will be 5.5 feet per second. Plate 12 represents a section following the centerline of channel, identifying elevations, grades, etc. for the inflow channel, control structure, and outflow channel.

The inflow channel will be a trapezoidal construction with 10 ft wide bottom, and a combination of 2:1 and 3:1 sideslopes. Plate 13 shows a typical inlet channel cross-section. The bottom and sides of the channel (to 7 ft elevation) will be covered with a non-woven geo-textile, followed by a 6 inch bedding layer of "dirty" gravel, and overlain by a 12 inch (minimum) layer of well graded D_{50} = 4" cobbles with fines (dirty) and seeded. In addition to providing bedding for the cobble channel lining, the dirty gravel will improve revegetation success in the channel, and substantially reduce the contribution of tremolite fibers from that portion of the channel. The dirty cobble lining should also improve reclamation success, further stabilizing the channel against storm events. The channel lining will be keyed into the sides of the channel as shown.

The lined portion of the channel will be excavated at a 2:1 slope, with the upper portions excavated at a 3:1 (refer to Plate 13). The concept behind this design is that the upper, unlined portions of the slope will have less potential for erosion prior to vegetation becoming established with a flatter slope. Also, vegetation will have a better success rate, and will become established quicker. Armoring the 2:1 slopes will prevent erosion and flood scour from occurring until vegetation becomes established. Should slope stability or other problems become evident during actual construction, the slopes will be flattened at that time.

5.4 CONTROL STRUCTURE

A control structure will be constructed through the tailings dam to control discharges from the reservoir, and into the outlet channel below the dam. The control structure will provide a method for safely reducing peak flows during major events while preserving the integrity of the dam and reducing the downstream impact. Our study of various control structures including open channels, concrete box culverts and metal pipe culverts suggests that the concrete box culvert provides the best method for controlling outflow while preserving the surge capacity of the impoundment for major storm events.

For the purpose of the conceptual study, we investigated two configurations for the box culvert control structure. These were twin 4 ft. by 6 ft. concrete box culverts (total open area 46.6 square feet), and a single 4 ft. by 8 ft. concrete box culvert (total open area 31.4 square feet). Both structures have an inflow elevation of 2900.0, and a 2% grade. Entrance construction will match adjacent contours.

Calculated peak outflow (26 feet elevation head) from the twin box culverts is 1080 cfs, and 744 cfs from the single box culvert. Design calculations for peak outflow are located in Appendix C. We then looked at the performance of these outlet structures under several flow conditions including the 100-year storm event and the 0.5 PMF event. A discussion of the performance of the systems under these conditions follows in Sections 5.4.1 and 5.4.2. Pertinent findings of this analysis are summarized in Table 5.1.

5.4.1 100-Year Event

Routing the 100-year, 24-hour event peak inflow of 460 cfs (Section 3.2.2) through the reservoir using a crest elevation (beginning of the inflow channel) of 2904.0, and the single 4' x 8' box culvert for outlet control, produced a peak discharge of 243 cfs and a maximum water surface elevation of 2905.4 at the outlet control structure. With this system,

Table 5.1 Flood routing parameters for various routing alternatives.

DESIGN FLOOD	PEAK FLOW (cfs)	CONTROL STRUCTURE	PEAK DISCHARGE[1] (cfs)	PEAK WATER ELEVATION (ft)
100-Year	460	twin 4' x 6'	228	2904.0
0.5 PMF	5838	as above	983	2922.6
0.5 PMF	5838	single 4' x 8'	731	2925.1
0.55 PMF	6320	twin 4' x 6'	1078	2925.9
0.66 PMF	7750	as above	2071[2]	2925.9
0.66 PMF	7700	single 5' x 9'	1860[3]	2925.9

1 Peak discharge from the proposed control structure.
2 Includes outflow from a 50 foot wide emergency spillway.
3 Includes outflow from a 35 foot wide emergency spillway.

virtually all of the impounded flows from a 100-year event would drain in <u>less</u> than 24 hours. This demonstrates that the surface water elevation of the impoundment will not rise significantly above the elevation of 2904.0 during a 100-year event if the twin box culverts are used to control outflow.

This is an important point. Should a 100-year event occur <u>immediately</u> before a 0.5 PMF event, the surface water elevation in the impoundment will remain at the proposed static water elevation of 2904.0. Because of this, routing/storage will begin at elevation 2904.0 rather than the existing emergency crest elevation of 2920.0 as outlined by the Montana Dam Safety regulations (State of Montana, 1989).

5.4.2 Probable Maximum Flood

Various percentages of the PMF event, beginning with a minimum of 0.5 PMF, were routed through the impoundment using representative outlet control scenarios, including with and without an emergency spillway. The results of the flood routing models are located in Appendix C.

Routing 0.5 PMF using the twin box culvert control (no emergency spillway) produced a peak discharge of 983 cfs, and a maximum water surface elevation of 2921.95, or slightly over 4 feet of freeboard remaining at the worst case. This routing was modeled using an initial water surface elevation of 2900, rather than the expected elevation of 2904. Final water surface elevations are estimated to be about 0.6 feet higher than the model results, or approximately 2922.55.

Routing 0.5 PMF using the single box culvert control (no emergency spillway) produced a peak discharge of 731 cfs at a maximum water surface elevation of 2925.1, or slightly less than 0.9 feet of freeboard. Again, a beginning elevation of 2900 was used, making the peak elevation slightly higher.

An event with a peak flow of 6320 cfs (approximately 0.55 PMF) was routed through the reservoir using the twin box culverts, and no emergency spillway. This event produced a peak outflow discharge of 1078 cfs, and a peak water surface elevation of 2925.9, or approximately 0.1 feet of freeboard.

A fourth model was completed to determine what peak flow the reservoir would safely handle with a 50 foot wide (2:1 sides) emergency spillway channel in conjunction with the twin box culverts. Setting the crest of the emergency spillway at elevation 2922.0 would allow an event of approximately 7750 cfs (0.66 PMF) through the impoundment without overtopping the dam. Maximum water elevation would be 2925.9.

The proposed location of the emergency spillway imposes constraints on the amount of available space to construct the spillway without affecting the existing USPS road. In order to avoid impacting the USPS road, the emergency spillway will be about 35 feet in width. A model was completed using a single box culvert and a 35 foot wide emergency spillway with a crest elevation of 2922.0. This system will allow an event of approximately 7700 cfs (0.66 PMF) through the impoundment with a peak discharge (combined culvert and emergency spillway) of approximately 1860 cfs, and a maximum water elevation of 2925.9.

The preferred control structure for this conceptual design is a single concrete box culvert with dimensions 4 ft by 8 ft, and an estimated total length of 120 feet (inlet to outlet). This structure will be less prone to blockage by debris and easier to maintain because of its large open area. The control structure will be placed in the east abutment of the tailings dam adjacent to the bedrock, and graded at 0.02 ft/ft, or 2%. Inlet (crest) elevation will be 2900.0 and outlet elevation approximately 2897.6. The control structure will outflow directly into the outflow channel (Section 5.5). Plate 14 shows a typical cross-section of the control structure at the centerline of the tailings dam.

Peak inflows from large events will enter the reservoir and be temporarily stored until discharged through the control structure at a greatly reduced rate. With a peak inflow of 5838 cfs at 0.5 PMF, and a maximum control structure discharge of 744 cfs at 26 feet elevation head (distance from free water surface to control structure crest elevation), outflows are reduced by greater than 85%.

Every precaution will be taken during final design and construction of the box culvert in the tailings dam to insure against failure and maintain the integrity of the dam. The box culvert will be bedded, backfilled, and compacted following strict specifications. Rip-rap in the apron approach to the inlet of the culvert will be upgraded to compensate for the acceleration of flow as it converges on the opening of the culvert. Provisions will be made for collection of debris before the culvert entry which could be substantial in a major flood. Constant on-site supervision will be provided by a Registered Professional Engineer.

With the reduction in peak discharge, the outflow channel will be considerably smaller and more stable, and flood impact to downstream areas will be greatly reduced as well.

5.5 OUTLET CHANNEL

The outiet or outflow channel will be constructed as part of the flood routing system, and will carry discharges from the reservoir control structure and returu them to the natural Rainy Creek channel downstream of the tallings dam. The outflow chamel, constructed on the east abutment, will consist of a heavlly armored channel in conjunction with a series of concrete grade control or drop structures. This type of construction will be both functional and aesthetically pleasing, and will quickly return the flows to Rainy Creek. Environruental disturbance will be kept to a minimum. Plate 8 shows the outflow channel in plan view.

The channel will begin at the outiet of the control structure (elevation 2897.6) and tie into the Rainy Creek channel at approximate elevation 2780, with a total length of about 1300 feet. Maximum gradient will be sllghtly over 0.04 ft/ft (4%), and will be adjusted to "fit" the existing terrain. Maximum drop height of the drop structures will be 12 feet. A section following the centerlIne of channel is found on Plate 12.

A typical cross-section of the of the channel will be trapezoidal construction with a 10 foot wide bottom and 2:1 sideslopes, heavlly armored with a minimum of 42 inches of rock rip-rap and underlain with a sand/gravel layer or a non-woven geotextlle filter cloth. The rip-rap will be well graded with a minimum size of 3 inches and a maximum size of 36 inches. A 12-foot wide access road will be constructed on the inside berm. Plate 15 shows a typical outflow section.

The grade control structures proposed wlll be straight reinforced concrete drop structures sirullar to the SCS Type C structures, with a maximum drop height of 12 feet. The drop structures wlll be placed to utilize existing terrain, and depending on foundation conditions encountered during final design field investigations, some modifications may be required. Approximate drop structure locations are shown on Plate 8. Appendix D contains a standard drawing for a Type C drop spillway.

Construction of the outiet will require a moderate amount of excavation in the hillside adjacent to the east abutment of the tallings dam. With the close proxiruity of bedrock, portions of the channel wlll be in weathered or unweathered bedrock, requiring drilling and blasting. Some modification of the designed sideslopes of the outflow channel may he made should final design field investigations indicate the presence of durable bedrock. The intent of the project is to allgn the channel to maximize the use of the existing terrain and minimize environruental disturbance.

5.6 EMERGENCY SPILLWAY

An eruergency rellef spillway will be constructed on the west abutment of the tailings dam, and work in conjunction with the main flood routing system to assure safe passage of storm events exceeding 0.5 PMF. It will be sized to provide additional flood routing capacity within the constraints of maintaining construction within the abutment area of the dam but without necessitating a relocation of the Forest Service road. The spillway is designed to prevent overtopping of the tailings dam for storms with peak inflows of approximately 7700 cfs or 0.66 PMF (35 ft.). Construction of this emergency spillway is not required by regulation, but as a method of improving public safety. Plate 8 shows the general location of the spillway.

The emergency relief spillway will be constructed adjacent to the west abutment of the tailings dam, and will terminate 300+ feet downstream of the centerline of the dam. The design will prevent damage to the dam by delaying release of the overflows until past the toe of the tailings dam.

A typical cross-section of the of the emergency relief spillway will be trapezoidal construction with a 30 to 35 foot wide bottom and 2:1 sideslopes, armored with a minimum of 36 inches of well graded rock rip-rap. Plate 16 shows a typical cross-section of the relief spillway.

5.7 REVEGETATION

Revegetation of the tailings impoundment area will stress the re-establishment of plant species for slope stabilization, reduced erosion, utilization of excess water, aesthetic enhancement and self perpetuating vegetation for wildlife. The re-vegetation plan includes grasses, forbes, shrubs, and trees.

A specific grass mix will be used for reseeding, with each specie selected for a particular advantage that will include fixing nitrogen, production of organic matter, early emergence for soil cover and species with deep root penetration to stabilize the soil and recover water from a greater soil thickness. The tailings impoundment area will be hydroseeded at approximately 24 lbs PLS/acre and 2000 lbs/acre organic mulch where soil conditions permit. The mulch will aid in erosion control, soil aeration, seed germination, seedling establishment, and organic material. Broadcast seeding will be done on the soft tailings materials which provide poor bearing capacity for hydromulching equipment. An 18-46-0 fertilizer will be applied concurrently to improve plant growth, color and vigor. All seeding will take place in the spring or early fall.

The lower, wetter portions of the tailings impoundment area are characteristic of riparian sites which naturally promote fast growing native species such as willow, aspen, alder, chokecherry, dogwood, current, serviceberry and rose wood. These species will be planted to utilize excess water on the area surrounding the tailings pond and the beach area. Larger-sized trees are subject to wind-throw and are not recommended for this specific location.

Smaller trees and shrubs will be planted along the side slopes of the tailings dam and excavated channels. Certain provisions of the dam safety law prohibit trees on the face of dams. However, since the impoundment will normally not be holding water at capacity, the use of trees to stabilize the dam face, particularly at the lower elevations, would appear to offer more benefits both aesthetically and structurally than leaving the face of the dam entirely barren. Shrubs will quickly establish a denser cover to protect tree seedlings and new grass. Roots, especially those of woody vegetation, help stabilize banks by holding soll, reduce sediruent flow and increases hydraulic resistance flow.

The coarse taillngs dump has already been reclaimed and revegetated. Dozer basins were installed as catchments for runoff in order to reduce the potential for erosion. The entire coarse tailings area was seeded with a mixture of grasses and clovers. Several thousand trees and native plant species have been planted randomly along the face of the coarse tailings dump and in the dozer basins.

The tailings impoundment is currently used by moose which forage for aquatic vegetation near its edges. The reestablishment of vegetation on other areas of the irupoundruent will encourage use by deer and elk which are also commonly seen in the area. The use of specific cultural treatments, proper seed selection and a diversity of woody plant material will aid in the re-establishment of vegetation which will have probable long-term soll stabilization and assist in the natural regeneration of a productive forest habitat.

5.8 STABILIZATION/EROSION CONTROL

An iruportant constituent of the flood routing system, and other (tallings inpoundment) closure activities will be reduction of erosion and long-terru stabilization. This is particularly unportant at this site as the tallings inpoundment and coarse taillngs dump are basically devoid of vegetation at the present, making them prone to erosion and other problems. W.R. Grace will exercise best management practices to reduce these concerns.

As described in the above sections, armoring of channels, revegetation, grade reduction (drop) structures, and other methods will be employed to reduce erosion in the flood routing systems. Cut slopes will be a maximum of 2:1 for long slopes, and 1 1/2:1 with spaced benches for road relocation and other lesser cuts. The emergency spillway vill be constructed to release flows past the toe of the dam, and the groin of the dam wlll be reinforced as necessary.

Fleetwood Creek, now located in a sideslope constructed drainage channel, will be returned to a more natural channel adjacent to the toe of the coarse tallings dump. The channel will be stabilized with natural materials where possible including vegetation, log structures, and other methods to improve geomorphic stabillty.

The reruaining impoundment wetland will improve surface water quallty through natural filtration and settllng.

5.9 OTHER CLOSURE ACTIVITIES

Other work that will be completed as part of the impoundment closure will be to remove the Rainy Creek diversion pipellne, remove and reclaim roads, regrade portions of the coarse tailings dump, and plant trees on the downstream face of the tallings dam (below the level of the tallings).

The final constmction activity for the ùnpoundment will be to demollsh the decant tower and plug its outflow piping with concrete.

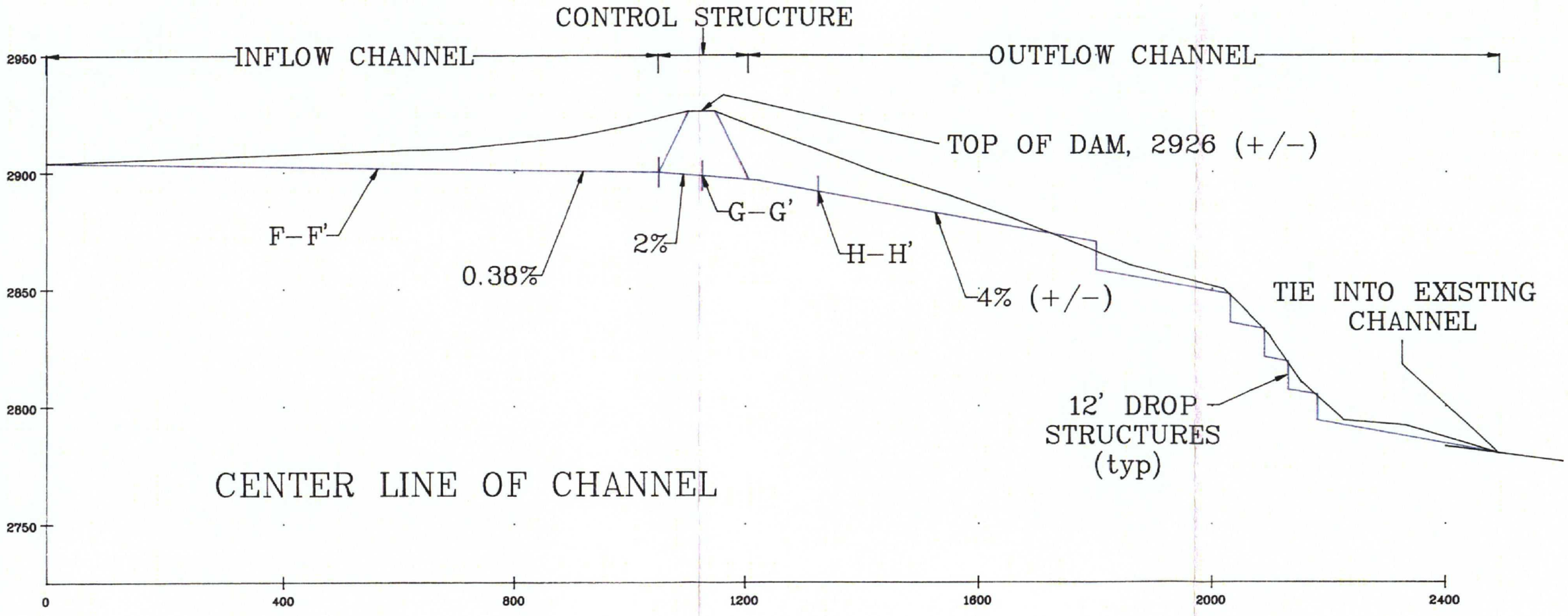


Plate 12. Section showing centerline of channel for the proposed flood routing system for the vermiculite tailings impoundment.

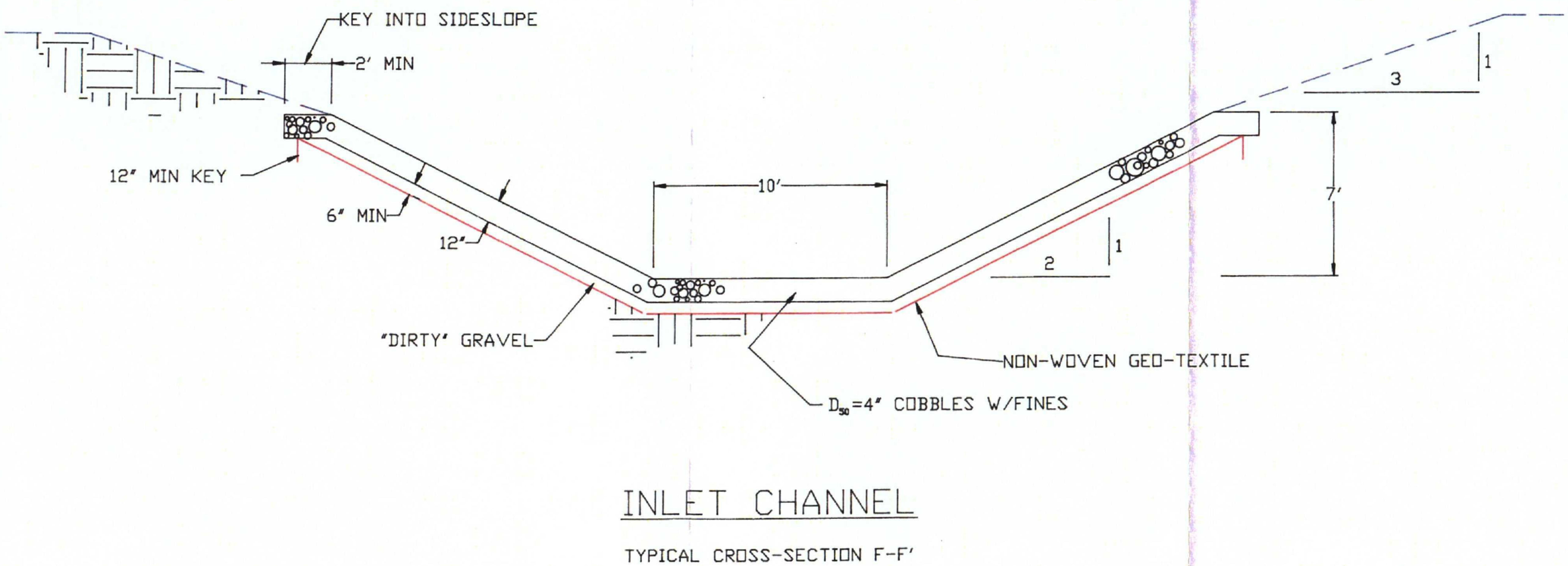


Plate 13. **Typical cross-section of the inflow channel for the proposed flood routing system.**

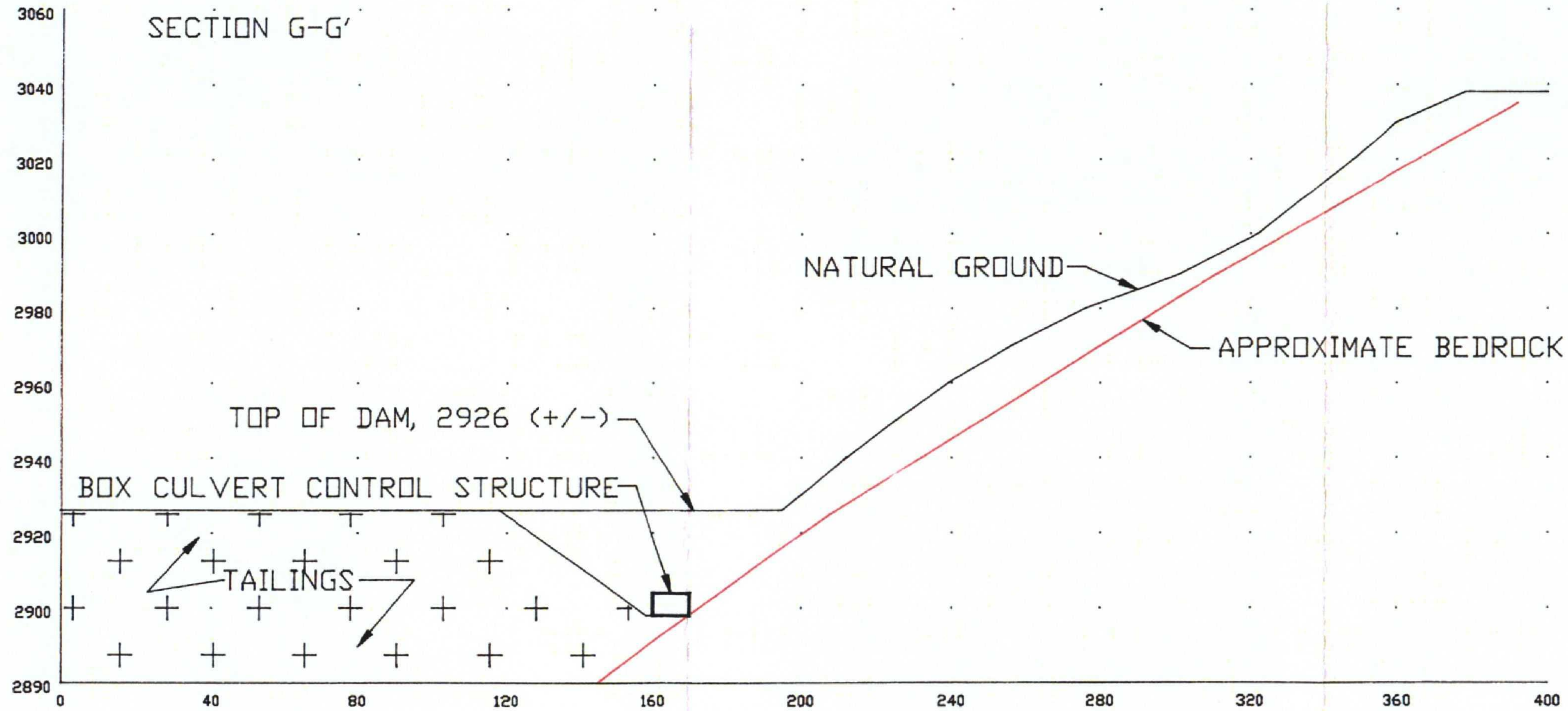


Plate 14. **Typical cross-section of the discharge control structure for the proposed flood routing system. Section taken from centerline of dam.**

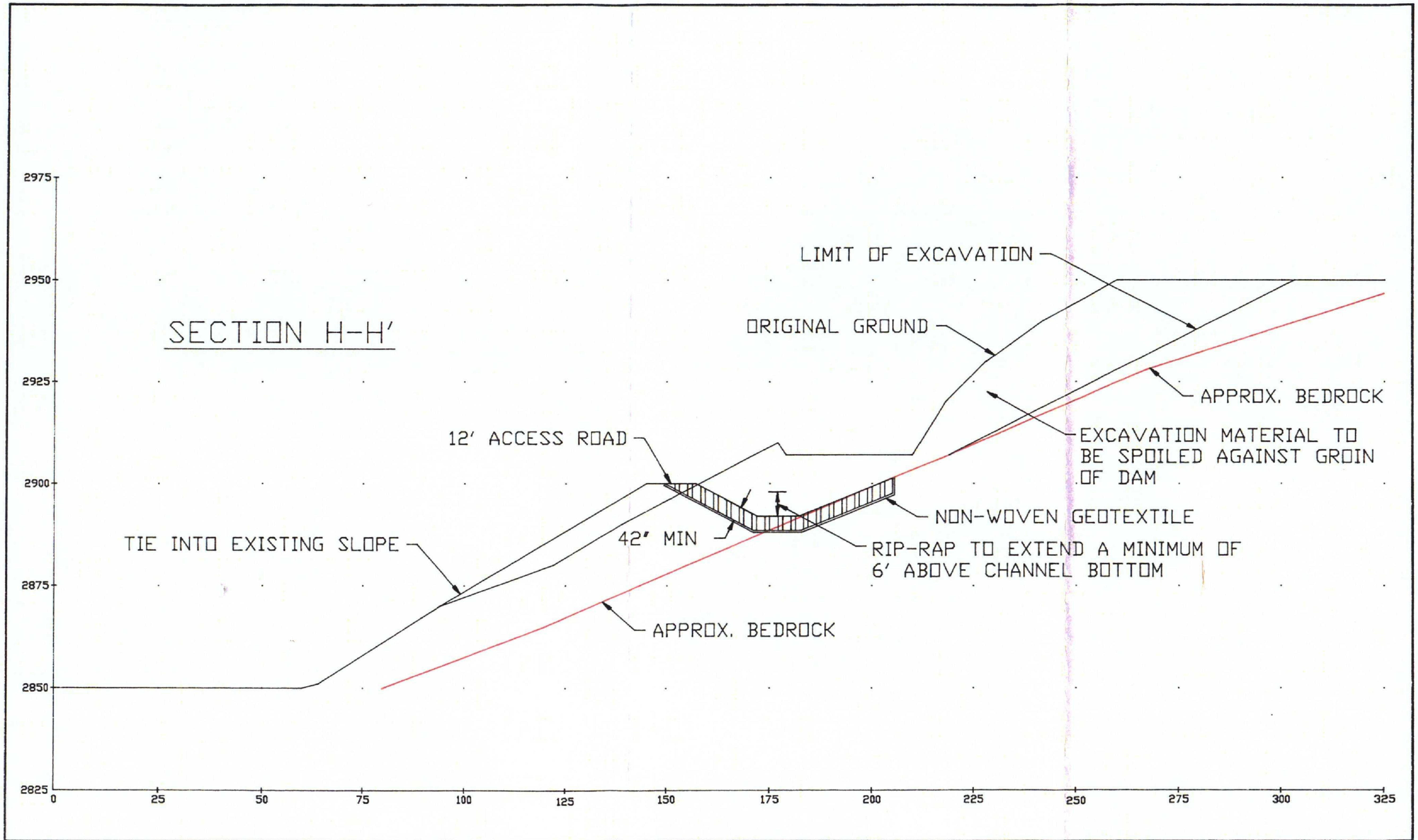


Plate 15. Typical cross-section of the outflow channel for the proposed flood routing system.

6.0 POST-CLOSURE CARE

6.1 POST-CLOSURE MANAGEMENT

W.R. Grace & Company is committed to proper management of the reclaimed mine property as long as it retains ownership of the property. Arrangements would be made for a fulltime custodian to look after the property. Part of the custodian's responsibilities will include periodic inspection of stream routing structures to assure proper operation and structural integrity.

W.R. Grace will close access to the upper mine property. However, situated next to the Forest Service access road, the tailings pond area will be accessible to the public. These areas will be posted for no trespassing. The custodian will provide security for this area to prevent unauthorized access to the property which will assure that initial revegetation efforts are not disturbed by recreational use. The custodian will also be responsible for coordination with regulatory agencies for ongoing monitoring activities.

6.2 WATER QUALITY MONITORING PROGRAM

A program of water quality monitoring was begun in the fall of 1991 by W.R. Grace to develop data regarding current water quality and to monitor the effects of closure activities on future water quality. This program is described in a document submitted to the Montana Department of State Lands, Water Quality Bureau (Hudson, 1991). The program calls for sampling and analysis of Rainy Creek, Fleetwood Creek, Carney Creek and discharges from the tailings impoundment. Monitoring will include heavy metals, although this should not be a problem for this particular mine, and asbestiform fibers. The monitoring will continue for a minimum of three years with provisions for additional monitoring depending on the results of the previous sampling

6.3 MAINTENANCE

The construction of channels for flood routing is not expected to be a solution without maintenance requirements. The recommended alternative is what we believe will offer the lowest maintenance requirements and least potential for catastrophic failures. The success of the closure in meeting these goals for the long-term depends on good maintenance practices. W.R. Grace is committed to this maintenance throughout its ownership of the property and will require that it be continued as a condition of any future sale of the property.

Areas which will require periodic inspection, on at least an annual basis, are the toe drain piping, box culvert outlet structure, and the constructed channels. Should the toe drains begin to fall and remedial action be indicated to prevent saturation and subsequent erosion of the dam foundations, W.R. Grace will implement appropriate corrective measures. A conceptual design for such remedial action has already been prepared by Harding Lawson Associates. Other structures may also require maintenance or reconstruction from time to time to assure continued functionallty according to intended design.

7.0 REFERENCES

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APPENDIX A

HYDROLOGIC MODELING RESULTS

W.R. GRACE HYDROGRAPH
FLEETWOOD CREEK
10-YEAR, 24-HOUR STORM (2.4 In.)

	INPUT DATA	TIME STEP (Hrs)	TIME (His)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (efs)
Total Pracip (Inches)	2.400	t.000	0.515	0.012	0.000	0.000	0.000
Total Ouration (Hrs)	24.000	2.000	1.030	0.026	0.000	0.000	0.000
		3.000	1.545	0.041	0.000	0.000	0.000
Area (Sq. miles)	3.900	4.000	2.061	0.035	0.000	0.000	0.000
Longest Run (Feel)	16370.000	5.000	2.579	0.070	0.000	0.000	0.000
Ave. Slope (%)	11.100	8.000	3.001	0.084	0.000	0.000	0.000
SCS Curve #	60.000	7.000	3.006	0.098	0.000	0.000	0.000
		8.000	4.121	0.115	0.000	0.000	0.000
Stonige 3 (Inches)	6.667	0.000	4.636	0.134	0.000	0.000	0.000
hitial Abetr. (Inches)	1.333	10.000	5.151	0.154	0.000	0.000	0.000
Time-consentration (Hrs)	2.576	11.000	9.657	0.173	0.000	0.000	0.000
Time-peak (Hrs)	1.803	12.000	6.182	0.192	0.000	0.000	0.000
Time-base (Hrs)	4.814	13.000	6.607	0.218	0.000	0.000	0.000
Duration (Hrs)	0.515	14.000	7.212	0.240	0.000	0.000	0.000
Iner. Preelp. (Inches)	0.052	15.000	7.727	0.264	0.000	0.000	0.000
		16.000	8.242	0.288	0.000	0.000	0.000
		17.000	0.757	0.336	0.000	0.000	0.000
		18.000	9.273	0.372	0.000	0.000	0.000
		10.000	0.788	0.413	0.000	0.000	0.000
		20.000	10.303	0.458	0.000	0.000	0.000
		21.000	10.818	0.523	0.000	0.000	0.000
		22.000	11.333	0.617	0.000	0.000	0.000
		23.000	11.848	0.929	0.000	0.010	0.883
		24.000	12.363	1.097	0.019	0 014	4.287
		23.000	12.879	1.819	0.033	0.011	11.824
		26.000	13.394	1.808	0.044	0.009	23.084
		27.000	13.909	1.958	0.093	0.008	34.836
		28.000	14.424	2.002	0.061	0.005	42.210
		20.000	14.938	2.038	0.067	0.006	40.137
		50.000	15.454	2 071	0.074	0.006	44.611
		31.000	15.960	2.100	0.078	0.006	42.476
		32.000	16.485	2 129	0.065	0.005	39.069
		33.000	17.000	2 155	0.080	0.008	37.712
		34.000	17.515	2 191	0.098	0.009	35.028
		35.000	18.030	2 213	0.102	0.004	34.688
		36.000	18.940	2 232	0.107	0.004	33.616
		37.000	19.060	2 251	0.111	0.004	32.267
		38.000	19.576	2.270	0.115	0.004	30.372
		38.000	20.091	2.287	0.110	0.003	28.474
		40.000	20.600	2.302	0.123	0.003	26.701
		41.000	21.121	2.316	0.126	0.005	25.088
		42.000	21.636	2.330	0.130	0.004	23.595
		43.000	22.181	2.345	0.133	0.004	22.301
		44.000	22.666	2 358	0.137	0.004	21.310
		45.000	23.181	2.374	0.140	0.004	20.625
		48.000	23.697	2.308	0.144	0.003	20.132
		47.000	24.212	2.400	0.147	0.000	19.609
		48.000	24.727	2.400	0.147	0.000	18.607
		49.000	25.242	2.400	0.147	0.000	16.698
		50.000	25.757	2.400	0.147	0.000	13.836
		51.000	20.272	2.400	0.147	0.000	10.561
		52.000	26.787	2.400	0.147	0.000	7.712
		53.000	27.309	2.400	0.147	0.000	5.510
		54.000	27.818	2.400	0.147	0.000	3.946
		55.000	28.333	2.400	0.147	0.000	2 842
		58.000	28.848	2.400	0.147	0.000	2.036
		97.000	29.383	2.400	0.147	0.000	1.466
		58.000	29.878	2.400	0.147	0.000	1.040
		69.000	30.393	2.400	0.147	0.000	0.740
		60.000	30.800	2.400	0.147	0.000	0.924
		61.000	31.424	2.400	0.147	0.000	0 370
		62.000	31.038	2.400	0.147	0.000	0.259
		60.000	38.454	2.400	0.147	0.000	0.176
		64.000	32.969	2.400	0.147	0.000	0.120
		65.000	33.484	2.400	0.147	0.000	0.078
		68.000	33.999	2.400	0.147	0.000	0.047
		67.000	34.519	2.400	0.147	0.000	0.025
		60.000	35.030	2.400	0.147	0.000	0.009
		69.000	35.545	2.400	0.147	0.000	0.000
		70.000	36.060	2.400	0.147	0.000	0.000

W.R. GRACE HYDROGRAPH
RAINY CREEK
10-YEAR, 24-HOUR STORM (2.4 In.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	2 400	1.000	0.709	0.012	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.417	0.034	0.000	0.000	0.000
		3.000	2.126	0.055	0.000	0.000	0.000
Area (Sq. miles)	5.900	4.000	2.834	0.077	0.000	0.000	0.000
Longest Run (Feet)	25870.000	9.000	3.543	0.098	0.000	0.000	0.000
Ave. Slope (%)	12.200	0.000	4.252	0.125	0.000	0.000	0.000
SCS Curve #	60.000	7.000	4.960	0.144	0.000	0.000	0.000
		0.000	5.609	0.173	0.000	0.000	0.000
Storage S (Inches)	0.667	9.000	0.377	0.204	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	7.086	0.240	0.000	0.000	0.000
Time-concentration (Hrs)	3.543	11.000	7.798	0.276	0.000	0.000	0.000
Time-peak (Hrs)	2.400	12.000	0.503	0.319	0.000	0.000	0.000
Time-base (Hrs)	6.622	13.000	9.212	0.353	0.000	0.000	0.000
Duration (Hrs)	0.700	14.000	9.921	0.413	0.000	0.000	0.000
Incr. Precip. (Inches)	0.071	15.000	10.629	0.487	0.000	0.000	0.000
		16.000	11.338	0.617	0.000	0.010	0.533
		17.000	12.046	1.591	0.010	0.023	3.614
		18.000	12.755	1.619	0.033	0.011	11.528
		19.000	13.464	1.808	0.044	0.013	25.239
		20.000	14.172	1.980	0.057	0.010	41.973
		21.000	14.881	2.038	0.067	0.000	59.838
		22.000	19.589	2.086	0.076	0.009	60.014
		23.000	16.298	2.129	0.085	0.008	69.494
		24.000	17.007	2.167	0.093	0.009	64.309
		25.000	17.715	2.191	0.098	0.007	61.614
		20.000	18.424	2.222	0.105	0.000	58.121
		27.000	19.132	2.251	0.111	0.007	54.338
		28.000	19.841	2.280	0.118	0.009	50.903
		29.000	20.590	2.302	0.123	0.009	48.280
		30.000	21.258	2.323	0.128	0.004	45.735
		31.000	21.967	2.338	0.131	0.005	43.092
		32.000	22.676	2.359	0.137	0.000	40.268
		33.000	23.384	2.381	0.142	0.005	37.818
		34.000	24.093	2.400	0.147	0.000	35.807
		35.000	24.801	2.400	0.147	0.000	33.788
		30.000	29.910	2.400	0.147	0.000	30.448
		37.000	28.219	2.400	0.147	0.000	25.439
		38.000	28.927	2.400	0.147	0.000	19.500
		39.000	27.636	2.400	0.147	0.000	14.248
		40.000	28.344	2.400	0.147	0.000	10.133
		41.000	29.053	2.400	0.147	0.000	7.251
		42.000	29.762	2.400	0.147	0.000	5.200
		43.000	30.470	2.400	0.147	0.000	3.733
		44.000	31.170	2.400	0.147	0.000	2.684
		45.000	31.887	2.400	0.147	0.000	1.897
		46.000	32.596	2.400	0.147	0.000	1.340
		47.000	33.305	2.400	0.147	0.000	0.997
		40.000	34.013	2.400	0.147	0.000	0.672
		49.000	34.722	2.400	0.147	0.000	0.465
		50.000	39.430	2.400	0.147	0.000	0.317
		61.000	36.139	2.400	0.147	0.000	0.214
		52.000	38.848	2.400	0.147	0.000	0.140
		33.000	37.596	2.400	0.147	0.000	0.080
		54.000	38.265	2.400	0.147	0.000	0.047
		99.000	38.974	2.400	0.147	0.000	0.017
		98.000	30.682	2.400	0.147	0.000	0.000
		57.000	40.391	2.400	0.147	0.000	0.000
		58.000	41.009	2.400	0.147	0.000	0.000
		59.000	41.808	2.400	0.147	0.000	0.000
		60.000	42.617	2.400	0.147	0.000	0.000
		61.000	43.228	2.400	0.147	0.000	0.000
		62.000	43.934	2.400	0.147	0.000	0.000
		63.000	44.642	2.400	0.147	0.000	0.000
		64.000	49.851	2.400	0.147	0.000	0.000
		69.000	46.060	2.400	0.147	0.000	0.000
		66.000	46.768	2.400	0.147	0.000	0.000
		87.000	47.477	2.400	0.147	0.000	0.000
		68.000	48.189	2.400	0.147	0.000	0.000
		89.000	48.094	2.400	0.147	0.000	0.000
		70.000	49.603	2.400	0.147	0.000	0.000

W.R. GRACE HYDROGRAPH
RAINY CREEK
100-YEAR, 24-HOUR STORM (3.4 in.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	3.400	1.000	0.700	0.017	0.000	0.000	0.000
Total Duration (Hrs)	24.000	2.000	1.417	0.048	0.000	0.000	0.000
		3.000	2.126	0.078	0.000	0.000	0.000
Area (Sq. miles)	5.900	4.000	2.834	0.109	0.000	0.000	0.000
Longest Run (Feet)	25870.000	8.000	0.943	0.139	0.000	0.000	0.000
Ave. Slope (%)	12.200	0.000	4.292	0.177	0.000	0.000	0.000
SCS Curve #	60.000	7.000	4.960	0.204	0.000	0.000	0.000
		0.000	0.669	0.249	0.000	0.000	0.000
Storage S (Inches)	6.007	0.000	0.377	0.289	0.000	0.000	0.000
Initial Abstr. (Inches)	1.333	10.000	7.086	0.340	0.000	0.000	0.000
Time-concentration (Hrs)	3.543	11.000	7.799	0.001	0.000	0.000	0.000
Time-peak (Hrs)	2.460	12.000	0.503	0.492	0.000	0.000	0.000
Time-base (Hrs)	0.622	10.000	9.212	0.000	0.000	0.000	0.000
Duration (Hrs)	0.709	14.000	9.921	0.585	0.000	0.000	0.000
Incr. Precip. (Inches)	0.100	15.000	10.629	0.690	0.000	0.000	0.000
		16.000	11.338	0.874	0.000	0.112	6.434
		17.000	12.046	2.254	0.112	0.084	31.209
		18.000	12.799	2.577	0.106	0.034	84.132
		19.000	13.464	2.689	0.220	0.037	159.174
		20.000	14.172	2.809	0.266	0.027	229.974
		21.000	14.881	2.887	0.294	0.024	262.377
		22.000	19.980	2.955	0.317	0.022	262.018
		23.000	10.298	0.016	0.339	0.020	244.487
		24.000	17.007	3.070	0.359	0.013	220.858
		29.000	17.719	3.104	0.372	0.017	190.888
		20.000	10.424	0.148	0.388	0.016	170.352
		27.000	19.132	0.180	0.404	0.010	159.606
		28.000	19.841	3.230	0.420	0.012	140.976
		29.000	20.950	0.261	0.432	0.012	131.924
		30.000	21.258	3.291	0.444	0.008	121.475
		31.000	21.967	0.312	0.493	0.012	111.794
		32.000	22.676	0.342	0.4A5	0.013	102.432
		80.000	23.384	3.373	0.478	0.011	94.523
		34.000	24.093	3.400	0.489	0.000	88.118
		00.000	24.601	3.400	0.489	0.000	82.102
		36.000	25.910	0.400	0.489	0.000	70.338
		37.000	20.219	0.400	0.489	0.000	60.938
		00.000	20.927	0.400	0.489	0.000	46.571
		39.000	27.636	0.400	0.489	0.000	33.759
		40.000	28.344	2.400	0.489	0.000	20.939
		41.000	20.053	0.400	0.489	0.000	17.109
		42.000	29.762	0.400	0.489	0.000	12.273
		43.000	30.470	0.400	0.480	0.000	2.786
		44.000	31.179	0.400	0.489	0.000	6.262
		48.000	31.887	0.400	0.489	0.000	4.459
		46.000	32.596	2.400	0.409	0.000	0.167
		47.000	80.808	0.400	0.480	0.000	2.241
		40.000	34.013	0.400	0.488	0.000	1.971
		40.000	34.722	0.400	0.489	0.000	1.087
		80.000	30.430	3 400	0.489	0.000	0.738
		91.000	30.139	3.400	0.489	0.000	0.407
		92.000	30.848	3.400	0.489	0.000	0.329
		53.000	37.556	0.400	0.489	0.000	0.207
		64.000	30.265	0.400	0.489	0.000	0.109
		95.000	38.974	0.400	0.489	0.000	0.039
		96.000	30.682	2.400	0.489	0.000	0.000
		87.000	40.391	0.400	0.489	0.000	0.000
		98.000	41.099	3.400	0.489	0.000	0.000
		99.000	41.808	3.400	0.489	0.000	0.000
		80.000	42.517	3.400	0.489	0.000	0.000
		61.000	43.229	3.400	0.489	0.000	0.000
		62.000	43.934	3.400	0.489	0.000	0.000
		80.000	44.642	3.400	0.489	0.000	0.000
		04.000	40.391	3.400	0.489	0.000	0.000
		09.000	40.060	0.400	0.489	0.000	0.000
		00.000	46.768	3.400	0.468	0.000	0.000
		67.000	47.477	2.400	0.488	0.000	0.000
		68.000	48.189	3.400	0.489	0.000	0.000
		69.000	48.894	2.400	0.486	0.000	0.000
		70.000	49.603	2.400	0.489	0.000	0.000

W.R. GRACE HYDROGRAPH
FLEETWOOD CREEK
100-YEAR, 24-HOUR STORM (3.4 in.)

	INPUT DATA	TIME STEP (Hrs)	TIME (Hrs)	CUMULATIVE PRECIP (Inches)	CUMULATIVE RUNOFF (Inches)	INCREMENTAL RUNOFF (Inches)	TOTAL FLOW (cfs)
Total Precip (Inches)	3.400	1.000	0.818	0.017	0.000	0.000	0.000
Total Duration (Hrs)	24.000	3.000	1.030	0.037	0.000	0.000	0.000
		3.000	1.545	0.058	0.000	0.000	0.000
Arsa (Sq. miles)	3.900	4.000	3.061	0.078	0.000	0.000	0.000
Longest Run (Feet)	16370.000	9.000	3.576	0.090	0.000	0.000	0.000
Ave. Slope (%)	11.100	6.000	3.001	0.119	0.000	0.000	0.000
SCS Curve #	60.000	7.000	3.606	0.139	0.000	0.000	0.000
		8.000	4.121	0.163	0.000	0.000	0.000
Storage S (Inches)	0.067	9.000	4.636	0.100	0.000	0.000	0.000
Initial Abetr. (Inches)	1.333	10.000	5.151	0.218	0.000	0.000	0.000
Time-eoncentration (Hrs)	3.576	11.000	3.687	0.249	0.000	0.000	0.000
Time-peak (Hrs)	1.803	13.000	0.182	0.272	0.000	0.000	0.000
Time-base (Hrs)	4.814	10.000	8.697	0.306	0.000	0.000	0.000
Duration (Hrs)	0.919	14.000	7.212	0.340	0.000	0.000	0.000
Incr. Pracip. (Inches)	0.073	15.000	7.727	0.374	0.000	0.000	0.000
		16.000	0.242	0.408	0.000	0.000	0.000
		17.000	0.757	0.476	0.000	0.000	0.000
		18.000	3.273	0.527	0.000	0.000	0.000
		19.000	9.788	0.589	0.000	0.000	0.000
		20.000	10.303	0.640	0.000	0.000	0.000
		21.000	10.818	0.741	0.000	0.000	0.000
		23.000	11.333	0.874	0.000	0.000	3.000
		20.000	11.848	1.316	0.000	0.148	8.958
		24.000	13.363	3.404	0.148	0.047	30.756
		23.000	13.879	3.977	0.196	0.034	70.213
		20.000	13.394	3.689	0.229	0.028	139.932
		27.000	13.909	3.771	0.299	0.021	191.041
		28.000	14.424	3.836	0.276	0.017	200.807
		29.000	14.939	3.887	0.294	0.016	194.918
		30.000	10.454	3.934	0.310	0.814	173.976
		31.000	15.969	3.979	3.324	0.015	193.814
		33.000	10.465	3.016	0.339	0.014	130.109
		33.000	17.000	3.093	0.393	0.019	120.709
		34.000	17.515	0.104	0.372	0.012	109.377
		39.000	10.080	3.139	0.383	0.010	101.139
		38.000	18.545	3.162	0.394	0.011	94.390
		37.000	19.060	3.189	0.404	0.011	87.029
		80.000	19.979	3.216	0.415	0.009	80.838
		39.000	20.091	3.240	0.424	0.000	74.278
		40.000	20.000	3.261	3.432	0.008	68.438
		41.000	21,121	3.281	0.440	0.008	63.338
		43.000	21.636	3.301	0.449	0.008	80.801
		40.000	23.1S1	3.332	0.457	0.008	94.047
		44.000	23.666	3.342	0.4S5	0.000	91.090
		49.000	23.181	3.363	0.474	0.008	49.740
		46.000	23.697	3.383	0.482	0.007	47.939
		47.000	24.212	3.400	0.489	0.000	46.378
		40.000	24.727	3.400	0.489	0.000	43.786
		49.000	29.242	3.400	3.489	0.000	39.153
		50.000	29.757	3.400	0.489	0.000	32.371
		91.000	20 272	3.400	3.489	0.000	24.684
		S3.000	20.787	3.400	3.489	0.000	10.018
		80.000	27.303	3.400	3.489	0.000	13.869
		94.000	27.818	3.400	0.489	0.000	9.219
		99.000	20.333	3.400	0.489	0.000	3.633
		56.000	20.848	3.400	0.489	0.000	4.747
		57.000	29.383	0.400	3.489	0.000	3.394
		80.000	29.878	3.400	0.469	0.000	3.429
		59.000	30.363	3.400	0.489	0.000	1.723
		60.000	30.009	3.400	0.489	0.000	1.219
		61.000	31.424	3.400	3.489	0.000	3.860
		63.000	31.939	3.400	0.489	0.000	3.601
		80.000	33.454	3.400	0.489	0.000	3.414
		64.000	33.969	3.400	3.489	0.000	3.279
		65.000	30.484	3.400	0.489	0.000	3.181
		66.000	33.983	3.400	3.480	0.000	0.110
		67.000	34.519	3.400	0.489	0.000	0.057
		68.000	30.030	8.400	0.466	0.000	0.020
		69.000	35.545	3.400	0.489	0.000	0.000
		70.000	36.060	3.400	0.489	0.000	0.000

W.R. GRACE HYDROGRAPH
TAILINGS IMPOUNDMENT WATERSHED
PMF STORM EVENT, 6-HOUR AUGUST THUNDERSTORM (10.7 in.)

Time (Hrs)	Incremental Rainfall (Inches)	Rainfall Rate (In/Hr)	Rainy Creek Flow (cfs)	Fleetwood Creek Flow (cfs)	Combined Flow (cfs)
0.000	0.000	0.000	0.000	0.000	0.000
0.500	0.200	0.400	0.000	0.000	0.000
1.000	0.300	0.600	5.000	6.000	11.000
1.500	0.500	1.000	22.000	35.000	57.000
2.000	2.200	4.400	73.000	169.000	238.000
2.500	4.500	9.000	280.000	447.000	727.000
3.000	1.000	2.000	1131.000	1394.000	2485.000
3.500	0.500	1.000	1782.000	3112.000	4894.000
4.000	0.500	1.000	3242.000	5278.000	8520.000
4.500	0.300	0.600	5582.000	5884.000	11466.000
5.000	0.300	0.600	6900.000	4776.000	11676.000
9.500	0.200	0.400	7330.000	3652.000	10982.000
6.000	0.200	0.400	6390.000	2917.000	9257.000
6.500	0.000	0.000	5012.000	2380.000	7392.000
7.000	0.000	0.000	4256.000	1956.000	6212.000
7.500	0.000	0.000	3560.000	1604.000	5164.000
8.000	0.000	0.000	2960.000	1350.000	4310.000
8.500	0.000	0.000	2606.000	1123.000	3729.000
9.000	0.000	0.000	2218.000	940.000	3198.000
9.500	0.000	0.000	1921.000	800.000	2721.000
10.000	0.000	0.000	1675.000	680.000	2389.000
10.500	0.000	0.000	1466.000	578.000	2044.000
11.000	0.000	0.000	1291.000	510.000	1601.000
11.500	0.000	0.000	1197.000	439.000	1636.000
12.000	0.000	0.000	1076.000	377.000	1453.000
12.900	0.000	0.000	963.000	322.000	1285.000
13.000	0.000	0.000	890.000	270.000	1160.000
13.500	0.000	0.000	804.000	213.000	1017.000
14.000	0.000	0.000	722.000	169.000	891.000
14.900	0.000	0.000	648.000	117.000	765.000
15.000	0.000	0.000	604.000	72.000	676.000
15.500	0.000	0.000	556.000	31.000	587.000
16.000	0.000	0.000	909.000	16.000	523.000
16.500	0.000	0.000	459.000	11.000	470.000
17.000	0.000	0.000	420.000	6.000	426.000
17.500	0.000	0.000	375.000	3.000	378.000
16.000	0.000	0.000	336.000		336.000
18.900	0.000	0.000	314.000		314.000
19.000	0.000	0.000	280.000		280.000
19.500	0.000	0.000	243.000		243.000
20.000	0.000	0.000	204.000		204.000
20.500	0.000	0.000	164.000		164.000
21.000	0.000	0.000	119.000		115.000
21.900	0.000	0.000	43.000		43.000
22.000	0.000	0.000	25.000		25.000
22.900	0.000	0.000	17.000		17.000
23.000	0.000	0.000	9.000		9.000
23.500	0.000	0.000	9.000		9.000
24.000	0.000	0.000	2.000		2.000

APPENDIX B

PROBABLE MAXIMUM FLOOD CALCULATIONS

→ USE METHOD FOR THUNDERSTORM PMP – pg 180 – 184 OF HMR 43 ←

1) DISTANCE FROM S.E. BORDER : ~ 488 miles

2) MAY : 71.5 %
JUNE : 79.1 %
JULY : 84.4 %
AUG : 88.6 % ← Use
SEPT : 74.9 %

NOTE: THIS METHOD IS TAKEN DIRECTLY FROM HMR 43 : "PROBABLE MAXIMUM PRECIPITATION, NORTHWEST STATES", U.S. WEATHER BUREAU, 1966

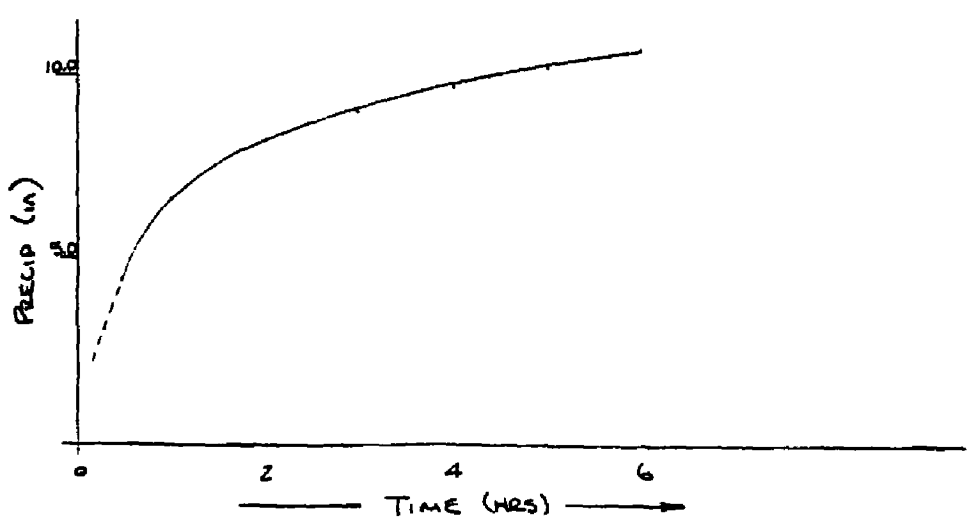
3) 2900' ELEVATION → ≤ 5000'

4) ½ HR : 6.0 in
1 HR : 8.0 in
2 HR 9.5 in
3 HR : 10.3 in
4 HR : 10.8 in
5 HR : 11.25 in
6 HR : 11.6 in

5) 1 HR : 84 %
3 HR : 87 %
6 HR : 92 %

6) 1 HR: (8.0)(0.84) = 6.7"
3 HR: (10.3)(0.87) = 9.0"
6 HR: (11.6)(0.92) = 10.7"

7) ½ HR: 4.5"
1 HR: 6.7"
2 HR 8.2"
3 HR: 9.0"
4 HR: 9.7"
5 HR: 10.2"
6 HR: 10.7"



7) CONT.

½	1	2	3	4	5	6
4.5	6.7	8.2	9.0	9.7	10.2	10.7
	6.7	1.5	.8	.7	.5	.5

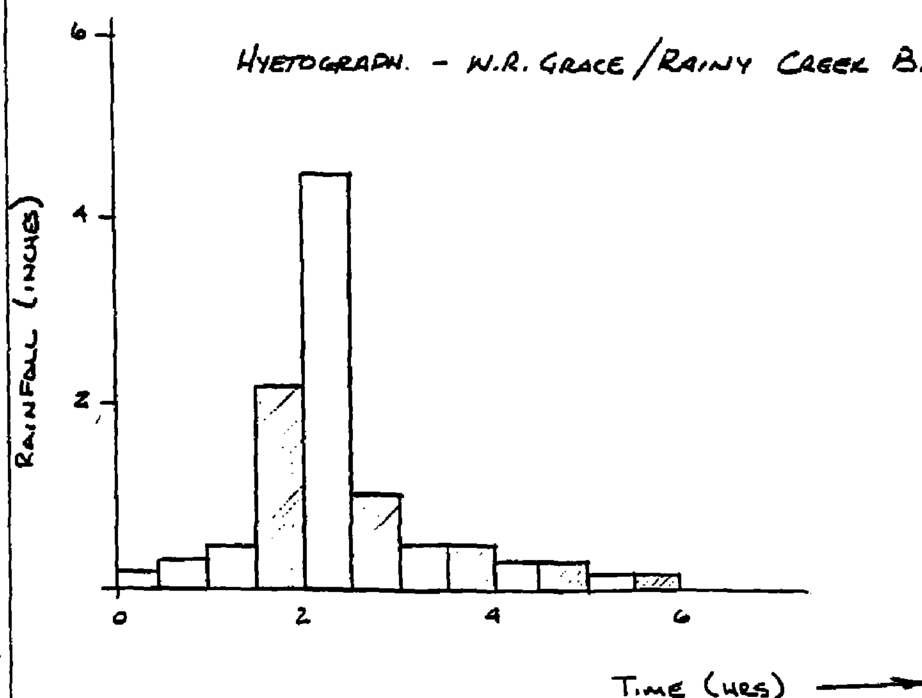
EXPAND FOR ½ HR DURATION: (USING RATIO 4.5/6.7 = 67%)

	①		②		③		④		⑤		⑥	
0	½	1	1½	2	2½	3	3½	4	4½	5	5½	6
0	4.5	6.7	7.7	8.2	8.7	9.2	9.5	9.7	10.0	10.2	10.5	10.7
4.5	2.2	1.0	0.5	0.5	0.5	0.3	0.2	0.3	0.2	0.3	0.2	
1	2	3	4	5	6	7	10	8	11	9	12	

8) SEQUENCE (½ HR DURATION)

0.2 0.3 0.5 2.2 4.5 1.0 0.5 0.5 0.3 0.3 0.2 0.2

NOTE: THESE VALUES WILL BE USED TO CALCULATE RUNOFF BY BUREAU OF RECLAMATION METHOD OUTLINED IN CHPT 4 OF "FLOOD HYDROLOGY MANUAL", BUREAU OF RECLAMATION, 1989.



→ CALCULATE RUNOFF FOR RAINY CREEK: ← *

RAINY CREEK: 5.9 SQ MI
25872 FT (4.9 mi) LONGEST REACH
12.2% (637'/mi) SLOPE

$$LAG = L_g = 26\,K_n\left(\frac{L\,L_{ca}}{S^{0.5}}\right)^{0.33}$$

where $L = 4.9$ mi
$L_{ca} = \sim 2.8$ mi
$S = 637'/\text{mi}$
$K_n = 0.17$ (estimated from Table 4.5)

$$\therefore\ L_g = 26(0.17)\left(\frac{(4.9)(2.8)}{\sqrt{637}}\right)^{0.33}$$

$$= 3.61 \text{ HRS}$$

DURATION $\sim L_g/5.5 = 3.61/5.5 = 0.65$ HR $= \sim 39$ minutes

CHECKING 15 to 20% OF LAG $0.15(3.61)60 = 32.5$ minutes ✓

→ USE 30 MINUTE DURATION ←

INFILTRATION LOSSES: SOIL TYPE 'B' · 0.15 TO 0.30 "/HR ULT. INFILTRATIO

PMP METHOD REQUIRES SATURATED SOILS W/ ULT. INFILTRATION → THIS AREA IS FOREST W/ EXC. GROUND COVER: USE 0.25"/HR

CALCULATE "S-graph"

$$q_{ult.}\ (\text{ultimate discharge}) = \frac{645.3 \times 5.9}{0.5} = 7615 \text{ hrs-cfs}$$

— NOTE: USE TABLE 4-16 (CASCADE AREA) FOR S-graph data

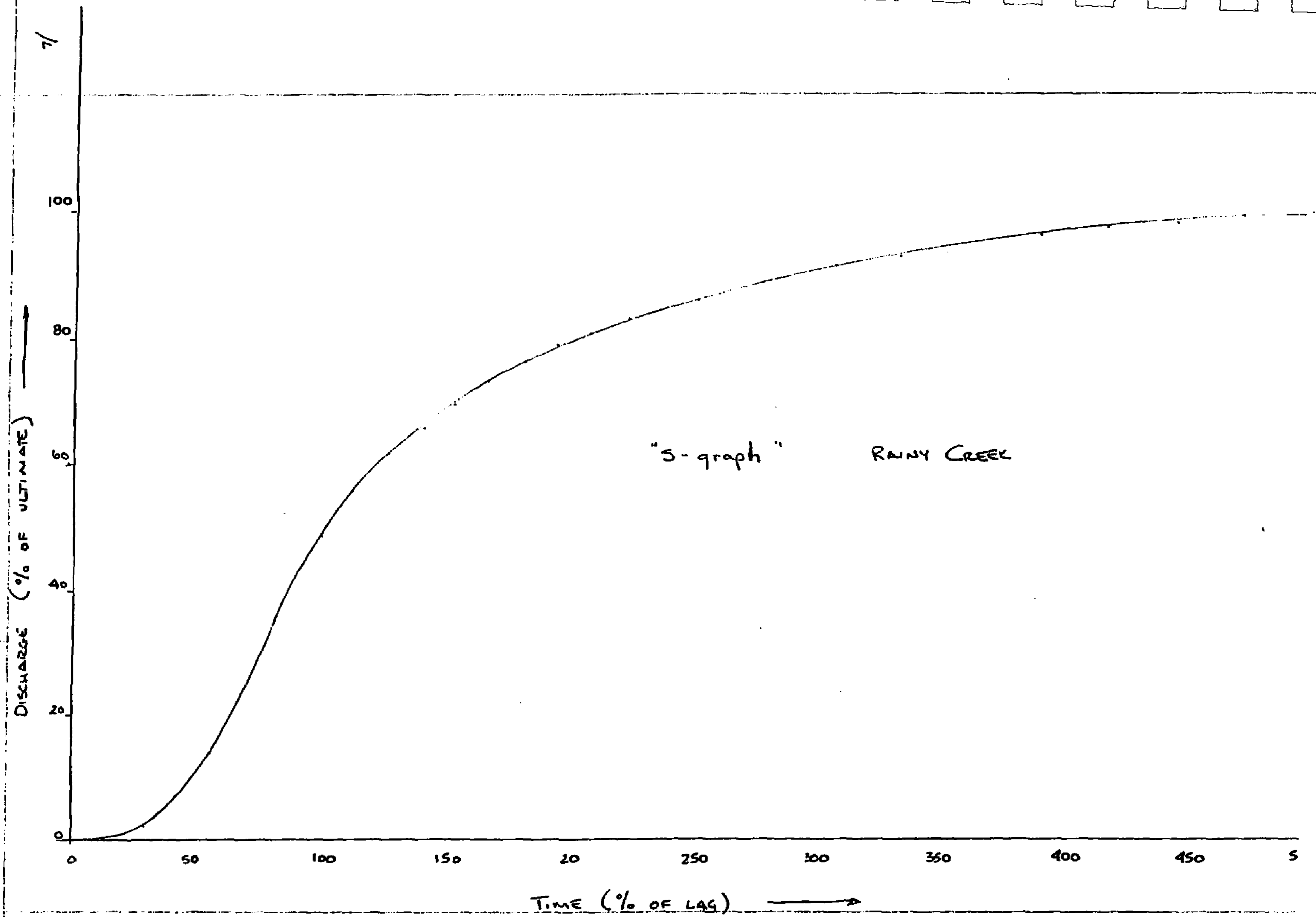
* NOTE: THIS METHOD IS TAKEN FROM CHPT 4, "FLOOD HYDROLOGY MANUAL", BUREAU OF REC, 1989.

TIME HRS	ACCUM RAINFALL	ACCUM. RUNOFF	INCREM RUNOFF	REVERSED INCREM. RUNOFF
0	0	0	0	.07
0.5	0.2	.08	.08	.08
1.0	0.5	.25	.17	.17
1.5	1.0	.63	.38	.18
2.0	3.2	2.70	2.07	.37
2.5	7.7	7.08	4.38	.38
3.0	8.7	7.95	.87	.87
3.5	9.2	8.33	.38	4.38
4.0	9.7	8.70	.37	2.07
4.5	10.0	8.88	.18	.38
5.0	10.3	9.05	.17	.17
5.5	10.5	9.13	.08	.08
6.0	10.7	9.20	.07	0

RAINY CREEK

TIME HRS	TIME (% Lg)	DISCHARGE (% ULT.)	Σ HYDROGRAPH ORDINATES (cfs)	UNIT HYDROGRAPH (cfs)	FLOOD HYDROGRA (cfs)
0	0	0	0	0	0
0.5	14	0.8	61	61	0
1.0	28	2.7	206	145	5
1.5	42	6.9	525	319	22
2.0	55	14.1	1074	549	73
2.5	69	25.5	1942	565	250
3.0	83	38.2	2909	967	1131
3.5	97	48.8	3716	807	1782
4.0	111	55.8	4249	533	3242
4.5	125	61.6	4691	442	5552
5.0	139	66.3	5049	358	6900
5.5	152	69.9	5323	274	7330
6.0	166	75.3	5582	259	6350
6.5	180	76.15	5799	217	5012
7.0	194	78.7	5993	194	4256
7.5	208	80.95	6164	171	3560
8.0	222	82.95	6317	153	2960
8.5	235	84.63	6445	128	2606
9.0	249	86.3	6572	127	2218
9.5	263	87.8	6686	114	1921
10.0	277	89.1	6785	59	1675
10.5	291	90.35	6880	93	1466
11.0	305	91.47	6965	85	1291

11.5	319	92.48	7042	77	1197
12.0	332	93.3	7108	66	1076
12.5	346	94.2	7172	64	963
13.0	360	94.96	7231	59	890
13.5	374	95.66	7285	54	804
14.0	388	96.3	7333	48	722
14.5	402	96.9	7378	45	648
15.0	416	97.4	7918	40	604
15.5	429	97.9	7452	34	556
16.0	443	98.3	7486	34	505
16.5	457	98.7	7516	30	459
17.0	471	99.0	7542	26	420
17.5	485	99.33	7564	22	375
18.0	498	99.56	7581	17	336
18.5	512	94.75	7596	15	314
19.0					280
19.5					243
20.0					204
					164
					115
					43
					25
					17
					9
					5
					2


7/
"S-graph"
RAINY CREEK
DISCHARGE (% OF ULTIMATE)
100
80
60
40
20
0
0
50
100
150
20
250
300
350
400
450
5
TIME (% OF LAG)

→ RUNOFF FOR FLEETWOOD CREEK ←

FLEETWOOD CREEK: AREA = 3.5 mi^2

L = 3.1 mi

L_{ca} = 1.2 mi

S = 5104 - 2900/3.1 = 711'/mi

$$Lag = 26 K_n \left(\frac{L L_{ca}}{S^{0.5}} \right)^{0.33}$$

$$= 26 (0.17) \left(\frac{(3.1)(1.2)}{\sqrt{711}} \right)^{0.33} = 2.29 \text{ HRS}$$

DURATION = $L_g / 5.5$ = $(2.29)(60)/5.5$ = 25 min → USE 30 min (TO BE CONS. W/ RAINY)

INFILTRATION: 0.25"/HR

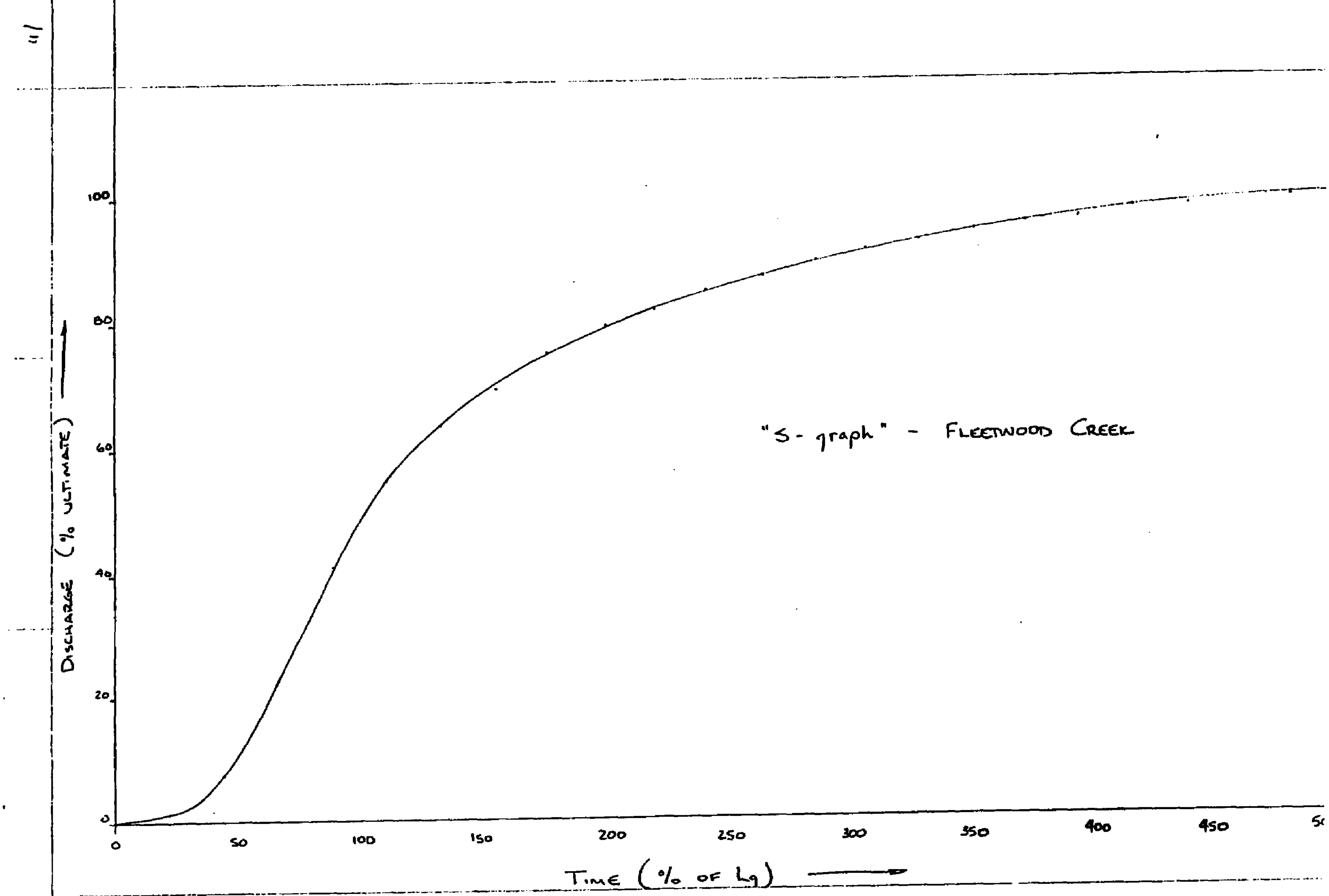
$$q_{ucr} = \frac{645.3 \times 3.5}{0.5} = 4517 \text{ hrs-cfs}$$

→ USE TABLE 4-16 (CASCADES)

FLEETWOOD CREEK

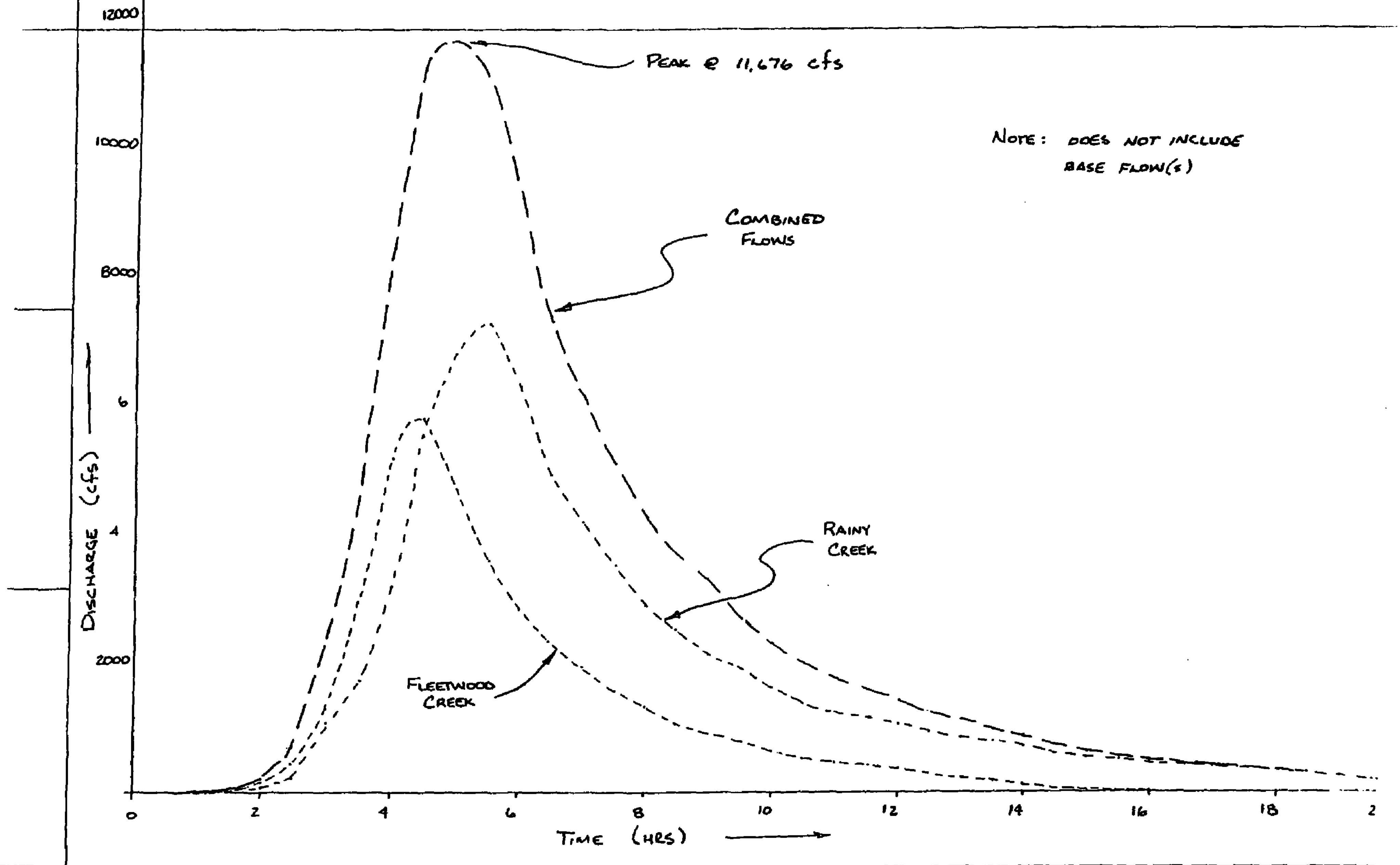
TIME HRS	TIME (%Lg)	DISCHARGE (% ULT)	Σ HYDROGRAPH ORDINATES (cfs)	UNIT HYDROGRAPH (cfs)	FLOOD HYDROGRAPH (cfs)
0	0	0	0	0	0
0.5	22	1.7	78	78	0
1.0	44	7.8	351	273	6
1.5	66	22.7	1024	673	35
2.0	87	41.3	1864	840	165
2.5	109	54.8	2476	612	447
3.0	131	63.7	2877	401	1354
3.5	153	70.2	3169	292	3112
4.0	175	75.2	3345	226	5278
9.5	197	79.2	3578	183	5889
5.0	218	82.4	3722	144	4776
5.5	240	85.2	3850	128	3652
6.0	262	87.7	3960	110	2917
6.5	284	89.8	4054	99	2380
7.0	306	91.5	4135	81	1956
7.5	328	93.1	4208	70	1604
8.0	349	94.35	4262	57	1350
8.5	571	95.8	4315	53	1123
9.0	393	96.5	4360	43	990
9.5	415	97.4	4399	39	800
10.0	437	98.1	4432	33	680
10.5	459	98.8	4461	29	578
11.0	480	99.2	4482	21	510

11.5	502	99.6	4500	18	439
12.0	524	99.9	4511	11	377
12.5	546	100.0	4517	7	322
13.0				0	270
13.5					213
14.0					169
14.5					117
15.0					72
					31
					18
					11
					6
					3


"S-graph" - FLEETWOOD CREEK
DISCHARGE (% ULTIMATE)
TIME (% OF Lg)
100
80
60
40
20
0
0
50
100
150
200
250
300
350
400
450

TIME (HRS)	RAINY CREEK (cfs)	FLEETWOOD CREEK (cfs)	COMBINED (cfs)
0	0	0	0
0.5	0	0	0
1.0	5	6	11
1.5	22	35	57
2.0	73	165	238
2.5	280	447	727
3.0	1131	1354	2485
3.3	1782	3112	4894
4.0	3242	5278	8520
4.5	5582	5884	11,466
5.0	6900	4776	11,676 ←
5.5	7330	3652	10,982
6.0	6350	2917	9267
6.5	5012	2380	7392
7.0	4256	1956	6212
7.5	3560	1604	5164
5.0	2960	1350	4310
8.5	2606	1123	3729
9.0	2218	940	3158
9.5	1921	800	2721
10.0	1675	680	2355
10.5	1466	578	2044
11.0	1291	510	1801

TIME	RAINY CREEK	FLEETWOOD CK	COMBINED
11.5	1197	439	1636
12.0	1076	377	1453
12.5	963	322	1235
13.0	890	270	1160
13.5	804	213	1017
14.0	722	169	891
14.5	648	117	765
15.0	604	72	676
15.5	556	31	587
16.0	505	18	523
16.5	459	11	470
17.0	420	6	426
17.5	375	3	378
18.0	336	0	336
18.5	314		314
19.0	280		280
19.5	243		245
20.0	204		204
20.5	164		164
21.0	115		115
21.5	43		43
22.0	25		25
22.5	17		17
23.0	9		9



Peak @ 11,676 cfs
Note: does not include base flow(s)
Combined Flows
Rainy Creek
Fleetwood Creek
12000
10000
8000
6
4
2000
Discharge (cfs)
0
2
4
6
8
10
12
14
16
18
2
Time (hrs)

APPENDIX C

CONTROL STRUCTURE and EMERGENCY SPILLWAY CALCULATIONS

CONTROL STRUCTURE

CULVERT RATING CURVE

SINGLE 4' X 8' CONCRETE BOX CULVERT

ELEVATION	INLET CONTROL			
	HW (ft)	HW/D	Q/B (cfs)	Q (cfs)
2900	0	0	0	0
2901	1.0	0.25	3.5	28
2902	2.0	0.50	7.2	58
2905	5.0	1.25	28	224
2907	7.0	1.75	40	320
2910	10.0	2.5	52	416
2915	15.0	3.75	68	544
2920	20.0	5.0	82	656
2926	26.0	6.5	93	744

CONTROL STRUCTURE

CULVERT RATING CURVE

TWIN 4' X 6' CONCRETE BOX CULVERTS

ELEVATION	INLET CONTROL				
	HW (ft)	HW/D	Q/B (cfs)	Q/Box (cfs)	Q_{Total} (cfs)
2900	0	0	0	0	0
2901	1.0	0.25	3.5	21	42
2902	2.0	0.50	7	42	84
2905	5.0	1.25	27	162	324
2907	7.0	1.75	38	228	456
2910	10.0	2.5	48	288	576
2915	15.0	3.75	65	390	780
2920	20.0	5.0	78	468	936
2926	26.0	6.5	90	540	1080

CONTROL STRUCTURE

CULVERT RATING CURVE

SINGLE 5' X 9' CONCRETE BOX CULVERT

ELEVATION	INLET CONTROL			
	HW (ft)	HW/D	Q/B (cfs)	Q (cfs)
2900	0	0	0	0
2901	1.0	0.25	3.5	31.5
2902	2.0	0.50	7	63
2905	5.0	1.25	28	252
2907	7.0	1.75	45	405
2910	10.0	2.5	52	468
2915	15.0	3.75	68	612
2920	20.0	5.0	82	738
2926	26.0	6.5	93	837

EMERGENCY SPILLWAY

STAGE-DISCHARGE

50 FT EMERGENCY SPILLWAY, CREST ELEVATION 2922.0

STAGE-DISCHARGE FOR EMERGENCY SPILLWAY			
ELEVATION	H_p	$Q_{High\ Flow}$	Q_{Total}
2922	---	---	990
2923	1.0	200	1223
2924	2.0	350	1386
2925	3.0	650	1709
2926	4.0	1000	2080

EMERGENCY SPILLWAY

STAGE-DISCHARGE

35 FT EMERGENCY SPILLWAY, CREST ELEVATION 2922.0

STAGE-DISCHARGE FOR EMERGENCY SPILLWAY			
ELEVATION	**H_P**	**$Q_{High\ Flow}$**	**Q_{Total}**
2922	---	---	1057
2923	1.0	140	1225
2924	2.0	245	1359
2925	3.0	455	1597
2926	4.0	700	1870

APPENDIX D

FLOOD ROUTING RESULTS

FLOOD ROUTING: 100-YEAR, 24-HOUR EVENT

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
100 YR 24 HR
LLB
12-15-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .2

CASE I EMERG. SPLWY. CURVE. CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 . WIDTH= 12
SIDE SLOPE= .001 . EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	458.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS.. PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD100.

TIME INT..HRS		INFLOW. CFS	S/T+O/2	OUTFLOW. CFS	EXIT VEL
INITIAL			1597.20	0.00	

PEAK
ELEV= 2900.00 STORAGE= 26.37
TIME= 0.00 -0.20 INFLOW= -1.73 S/T O/2= 1595.47
TOTAL SPLWY DIS= 0.00 CTR= 0
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 0.00 EMRG EXIT VEL= 0.00

0.80	-1.00	3.92	1599.91	0.24	0.69
1.80	-2.00	5.86	1621.99	2.17	1.68
2.80	-3.00	5.99	1637.91	3.57	2.05
3.80	-4.00	6.00	1648.11	4.46	2.24
4.80	-5.00	6.00	1654.56	5.03	2.35
5.80	-6.00	6.00	1658.65	5.38	2.42
6.80	-7.00	6.00	1661.23	5.61	2.46
7.80	-8.00	6.00	1662.86	5.75	2.48
8.80	-9.00	6.00	1663.89	5.84	2.50
9.80	-10.00	6.06	1664.60	5.91	2.51
[illegible]	[illegible]	[illegible]	[illegible]	12.30	3.36

12.80	-13.00	316.56	3483.72	140.53	8.91
13.80	-14.00	286.64	4159.90	194.24	10.14
14.80	-15.00	258.97	4477.29	219.45	10.65
15.80	-16.00	234.42	4577.95	227.45	10.81

PEAK

ELEV= 2903.80 STORAGE= 73.84
TIME= 16.00 -16.20 INFLOW= 230.43 S/T O/2= 4580.93
TOTAL SPLWY DIS= 227.69 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 227.69 EMRG EXIT VEL= 10.81

16.80	-17.00	219.14	4562.80	226.25	10.78
17.80	-18.00	207.91	4503.14	221.51	10.69
18.80	-19.00	195.51	4411.02	214.19	10.55
19.80	-20.00	183.75	4298.89	205.28	10.37
20.80	-21.00	176.84	4188.19	196.49	10.19
21.80	-22.00	167.59	4078.97	187.81	10.01
22.80	-23.00	162.72	3978.78	179.86	9.84
23.80	-24.00	156.54	3889.10	172.73	9.68
24.80	-25.00	151.36	3807.64	166.26	9.53
25.80	-26.00	104.43	3639.89	152.94	9.22
26.80	-27.00	40.20	3247.26	121.75	8.42
27.80	-28.00	15.30	2822.50	88.01	7.39
28.80	-29.00	8.32	2482.82	65.67	6.57
29.80	-30.00	6.46	2224.24	49.29	5.86
30.80	-31.00	6.06	2034.84	36.26	5.18
31.80	-32.00	6.06	1905.39	26.29	4.56
32.80	-33.00	6.06	1818.75	19.42	4.04
33.80	-34.00	6.06	1762.71	14.50	3.59
34.80	-35.00	6.06	1727.28	11.40	3.26
35.80	-36.00	6.06	1704.88	9.44	3.03
36.80	-37.00	6.06	1690.72	8.20	2.86
37.80	-38.00	6.06	1681.77	7.41	2.75
38.80	-39.00	6.06	1676.11	6.92	2.67
39.80	-40.00	6.06	1672.54	6.80	2.62
40.80	-41.00	6.06	1670.27	6.40	2.59
41.80	-42.00	6.06	1668.84	6.28	2.57

TOTAL VOLUME EMERG SPLWY FLOW= 273.43 AF
TOTAL VOLUME OF HYD ROUTED= 274.50 AF

FLOOD ROUTING: 100-YEAR, 24-HOUR EVENT

SINGLE 4' X 8' BOX CULVERT

RESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
100 YR 24 HR
LLB
03-17-1992

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .25

CASE I EMERG. SPLWY. CURVE. CREST LENGTH = 20 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 . WIDTH= 8
SIDE SLOPE= .001 , EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:

CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	O/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	8.00	0.00	100
2900.50	0.00	0.00	1.75	8.00	14.00	100
2901.00	0.00	0.00	3.50	8.00	28.00	100
2902.00	0.00	0.00	7.50	8.00	60.01	100
2904.00	0.00	0.00	21.17	8.00	169.38	100
2906.00	0.00	0.00	34.00	8.00	272.11	100
2907.00	0.00	0.00	40.00	3.00	320.15	100
2910.00	0.00	0.00	52.00	8.00	416.23	100
2915.00	0.00	0.00	68.00	8.01	544.36	100
2920.00	0.00	0.00	82.00	8.01	656.49	100
2926.00	0.00	0.00	93.00	8.01	744.60	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1277.76	0.00	100
2900.50	30.19*	1468.07	14.00	100
2901.00	34.52*	1684.63	28.00	100
2902.00	45.13*	2214.09	60.01	100
2904.00	77.11*	3816.60	169.38	100
2906.00	131.70	6510.34	272.11	100
2907.00	158.86*	7848.92	320.15	100
2910.00	278.70	13697.20	416.23	100
2915.00	549.70	26877.66	544.36	100
2920.00	870.70	42470.13	656.49	100
2926.00	1301.50	63364.90	744.60	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .25 HRS., PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD100

TIME INT..HRS		INFLOW. CFS	S/T+O/2	OUTFLOW, CFS	EXIT VEL
INITIAL			1277.76	0.00	
0.75	-1.00	3.71	1281.88	0.30	0.90
[illegible]	[illegible]	5.78	1300.05	1.64	1.77

[illegible]	[illegible]	[illegible]	[illegible]		
3.75	-4.00	6.00	1326.98	3.62	2.43
4.75	-5.00	6.00	1335.49	1.25	2.59
5.75	-6.00	6.00	1341.77	4.71	2.69
6.75	-7.00	6.00	1346.39	5.05	2.77
7.75	-8.00	6.00	1349.79	5.30	2.83
8.75	-9.00	6.00	1352.30	5.48	2.86
9.75	-10.00	6.16	1354.30	5.63	2.89
10.75	-11.00	75.36	1460.52	13.45	4.10
11.75	-12.00	360.15	2349.24	69.23	7.90
12.75	-13.00	435.14	3648.53	157.91	10.98
13.75	-14.00	378.10	4548.77	197.31	12.00
14.75	-15.00	331.31	5114.21	218.87	12.51
15.75	-16.00	294.81	5448.27	231.61	12.80
16.75	-17.00	271.94	5627.55	238.45	12.95
17.75	-18.00	255.95	5716.03	241.82	13.02

PEAK

ELEV= 2905.42 STORAGE= 115.92

TIME= 18.50 -18.75 INFLOW= 243.31 S/T O/2= 5731.91

TOTAL SPLWY DIS= 242.43 CTR= 100

PRIN Q= 0.00 CHUTE Q= 0.00

EMRG Q= 242.43 EMRG EXIT VEL= 13.04

18.75	-19.00	239.40	5728.89	242.31	13.04
19.75	-20.00	224.04	5880.19	240.45	12.99
20.75	-21.00	211.46	5594.64	237.19	12.92
21.75	-22.00	202.75	5480.32	232.83	12.83
22.75	-23.00	196.00	5349.53	227.84	12.72
23.75	-24.00	188.19	5210.61	222.55	12.60
24.75	-25.00	181.49	5066.15	217.04	12.47
25.75	-26.00	128.36	4827.25	207.93	12.26
26.75	-27.00	51.33	4330.29	188.97	11.80
27.75	-28.00	18.92	3721.97	162.93	11.12
28.75	-29.00	9.32	3176.62	125.70	10.02
29.75	-30.00	6.71	2749.61	96.56	9.02
30.75	-31.00	6.11	2423.63	74.31	8.12
31.75	-32.00	6.11	2177.51	57.80	7.35
32.75	-33.00	6.11	1988.76	46.39	6.73
33.75	-34.00	6.11	1841.68	37.50	6.18
34.75	-35.00	6.11	1727.07	30.57	5.69
35.75	-36.00	6.11	1637.89	24.98	5.25
36.75	-37.00	6.11	1569.42	20.55	4.86
37.75	-38.00	6.11	1517.01	17.17	4.52
38.75	-39.00	6.11	1476.90	14.57	4.23
39.75	-40.00	6.11	1446.39	12.41	3.97
40.75	-41.00	6.11	1423.85	10.75	3.75
41.75	-42.00	6.11	1407.25	9.53	3.57
42.75	-43.00	0.00	1388.91	8.18	3.36

TOTAL VOLUME EMERG SPLWY FLOW= 344.68 AF

TOTAL VOLUME OF HYD ROUTED= 346.80 AF

FLOOD ROUTING: 0.5 PMF EVENT

TWIN 4' X 6' BOX CULVERTS

```
.ESERVOIR ROUTING PROG.  (RES.BAS)  SMH.3-06-87

   WR GRACE DAM
   .5PMP
   LLB
   12-14-1991

 INPUT CONTROLS:
 NO OF STORAGE CURVE POINTS=  7 .  DELTA T=  .2

  CASE I  EMERG. SPLWY. CURVE. CREST LENGTH = 500 FT
 EARTH EMERG. SPILLWAY:  CREST EL.=  2900 . WIDTH=  12 '
       SIDE SLOPE=  .001 . EXIT SLOPE=  .04


  SPILLWAY DISCHARGE CURVES:
...........................
 CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
      500=ES CURVE IS EXCEEDED, Q'S ARE EXTRAPOLATED
```

ELEV	PRIN. Q	CHUTE Q	O/FT	EFF. W.	TOT. Q	CTR
			EMERG. SPLWY. VALUES			
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	456.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

```
 **** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****


 INITIAL ROUTING ELEV=  2900.00   STORAGE=      26.40
 ACTUAL DELTA T ROUTING INTERVAL=  .2  HRS.,  PRINTOUT INTERVAL=  .1  HRS.
 INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD55.PMP
```

TIME INT..HRS		INFLOW. CFS	S/T+O/2	OUTFLOW. CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.40	-0.60	3.41	1598.88	0.15	0.57
0.60	-0.80	5.89	1604.62	0.65	1.04
0.80	-1.00	7.02	1610.99	1.21	1.33
1.00	-1.20	7.55	1617.33	1.76	1.55
1.20	-1.40	7.79	1623.36	2.29	1.72
1.40	-1.60	7.90	1628.97	2.78	1.86
1.60	-1.80	7.96	1634.14	3.24	1.97
1.80	-2.00	7.98	1638.89	3.65	2.07
2.00	-2.20	7.99	1643.22	4.03	2.15
2.20	-2.40	8.00	1647.18	4.38	2.23
2.40	-2.60	8.00	1650.80	4.70	2.29
2.60	-2.80	8.00	1654.10	4.99	2.34
2.80	-3.00	8.00	1657.12	5.25	2.39
3.00	-3.20	8.00	1659.87	5.49	2.44
3.20	-3.40	8.00	1662.37	5.71	2.48
3.40	-3.60	8.00	1664.66	5.91	2.51
3.60	-3.80	8.00	1666.75	6.10	2.54
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

4.20	-4.40	8.00	1671.98	6.55	2.62
4.40	-4.60	8.00	1673.42	6.68	2.64
4.60	-4.80	8.00	1674.74	6.80	2.65
4.80	-5.00	8.00	1675.95	6.90	2.67
5.00	-5.20	8.00	1677.05	7.00	2.68
5.20	-5.40	8.00	1678.05	7.09	2.70
5.40	-5.60	8.00	1678.96	7.17	2.71
5.60	-5.80	8.00	1679.80	7.24	2.72
5.80	-6.00	8.00	1680.56	7.31	2.73
6.00	-6.20	8.01	1681.26	7.37	2.74
6.20	-6.40	8.04	1531.94	7.43	2.75
6.40	-6.60	8.19	1682.70	7.49	2.76
6.60	-6.80	8.77	1683.98	7.60	2.78
6.80	-7.00	10.35	1686.72	7.85	2.81
7.00	-7.20	13.74	1692.61	8.36	2.88
7.20	-7.40	19.77	1704.02	9.36	3.02
7.40	-7.60	29.09	1723.75	11.09	3.23
7.60	-7.80	42.21	1754.87	13.82	3.52
7.80	-8.00	59.39	1800.44	17.81	3.90
8.00	-8.20	80.74	1863.38	23.05	4.32
8.20	-8.40	106.27	1946.60	29.46	4.77
8.40	-8.60	135.87	2053.01	37.66	5.26
8.60	-8.80	169.87	2185.22	46.81	5.74
8.80	-9.00	209.31	2347.72	57.11	6.22
9.00	-9.20	256.38	2546.98	89.74	6.73
9.20	-9.40	313.96	2791.20	85.52	7.31
9.40	-9.60	385.43	3091.11	109.35	8.06
9.60	-9.80	475.67	3457.43	138.44	8.86
9.80	-10.00	597.13	3916.11	174.88	9.73
10.00	-10.20	790.91	4532.14	223.81	10.74
10.20	-10.40	1140.91	5440.24	272.70	11.62
10.40	-10.60	1738.37	6914.91	336.11	12.63
10.60	-10.80	2589.74	9168.53	429.72	13.94
10.80	-11.00	3554.47	12293.28	496.44	14.78
11.00	-11.20	4403.85	16200.69	560.62	15.50
11.20	-11.40	4954.91	20594.98	618.83	16.12
11.40	-11.60	5144.53	25120.68	674.83	16.69
11.60	-11.80	5015.65	29461.50	728.54	17.21
11.80	-12.00	4678.04	33411.00	777.41	17.66
12.00	-12.20	4253.34	36886.93	806.27	17.92
12.20	-12.40	3824.35	39905.00	830.44	18.14
12.40	-12.60	3429.79	42504.36	851.25	18.32
12.60	-12.80	3084.40	44737.51	869.13	18.47
12.80	-13.00	2790.65	46659.04	884.51	18.60
13.00	-13.20	2544.68	48319.20	897.80	18.71
13.20	-13.40	2340.83	49762.23	909.36	18.81
13.40	-13.60	2174.31	51027.18	919.49	18.89
13.60	-13.80	2039.95	52147.64	928.46	18.96
13.80	-14.00	1931.21	53150.40	936.47	19.03
14.00	-14.20	1840.70	54054.62	941.46	19.07
14.20	-14.40	1761.56	54874.71	945.98	19.11
14.40	-14.60	1688.89	55617.63	950.08	19.14
14.60	-14.80	1620.32	56287.87	953.78	19.17
14.80	-15.00	1555.92	56890.01	957.10	19.20
15.00	-15.20	1496.93	57429.84	960.07	19.22
15.20	-15.40	1444.30	57914.07	962.74	19.24
15.40	-15.60	1398.18	58349.51	965.14	19.26
15.60	-15.80	1358.21	58742.58	967.31	19.28
15.80	-16.00	1324.07	59099.34	969.28	19.29
16.00	-16.20	1295.51	59425.57	971.08	19.31
16.20	-16.40	1271.71	59726.20	972.74	19.32
16.40	-16.60	1251.14	60004.61	974.27	19.33
16.60	-16.80	1232.01	60262.35	975.69	19.34
16.80	-17.00	1212.81	60499.46	977.00	19.36

17.40	-17.60	1147.33	61075.36	980.18	19.38
17.60	-17.80	1123.65	61218.84	980.97	19.39
17.80	-18.00	1100.68	61338.55	981.63	19.39
18.00	-18.20	1079.50	61436.43	982.17	19.40
18.20	-18.40	1060.39	61514.66	982.50	19.40
18.40	-18.60	1042.60	61574.66	982.93	19.40
18.60	-18.80	1024.93	61616.66	983.16	19.40
18.80	-19.00	1006.36	61639.86	983.29	19.40
19.00	-19.20	986.79	61643.37	983.31	19.40

PEAK
ELEV= 2921.95 STORAGE= 1010.77
TIME= 19.00 -19.20 INFLOW= 986.79 S/T 0/2= 61643.37
TOTAL SPLWY DIS= 983.31 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG O= 983.31 EMRG EXIT VEL= 19.40

19.20	-19.40	967.55	61627.61	983.22	19.40
19.40	-19.60	950.95	61595.34	983.04	19.40
19.60	-19.80	938.53	61550.83	982.80	19.40
19.80	-20.00	929.77	61497.80	982.51	19.40
20.00	-20.20	922.51	61437.80	982.17	19.40
20.20	-20.40	914.23	61369.86	981.80	19.39
20.40	-20.60	903.26	61291.32	981.37	19.39
20.60	-20.80	889.28	61199.23	980.86	19.39
20.80	-21.00	873.44	61091.82	980.27	19.38
21.00	-21.20	857.66	60969.21	979.59	19.38
21.20	-21.40	843.50	60833.12	978.84	19.37
21.40	-21.60	831.80	60686.08	978.03	19.36
21.60	-21.80	822.81	60530.86	977.17	19.36
21.80	-22.00	816.20	60369.89	976.29	19.35
22.00	-22.20	811.15	60204.75	975.37	19.34
22.20	-22.40	806.37	60035.74	974.44	19.33
22.40	-22.60	800.46	59861.76	973.48	19.33
22.60	-22.80	792.52	59680.80	972.49	19.32
22.80	-23.00	782.76	59491.08	971.44	19.31
23.00	-23.20	772.23	59291.86	970.34	19.30
23.20	-23.40	762.14	59083.66	969.19	19.29
23.40	-23.60	753.46	58367.93	968.00	19.28
23.60	-23.80	746.63	58646.56	966.78	19.27
23.80	-24.00	741.53	53421.30	965.54	19.26
24.00	-24.20	737.48	58193.24	964.28	19.25
24.20	-24.40	733.22	57962.18	963.01	19.24
24.40	-24.60	726.14	57725.30	961.70	19.23
24.60	-24.80	710.90	57474.50	960.32	19.22
24.80	-25.00	679.79	57193.98	958.77	19.21
25.00	-25.20	626.10	56861.30	956.94	19.20
25.20	-25.40	549.84	56454.20	954.69	19.18
25.40	-25.60	459.46	55958.97	951.96	19.16
25.60	-25.80	366.79	55373.80	948.74	19.13
25.80	-26.00	282.04	54707.11	945.06	19.10
26.00	-26.20	211.59	53973.64	941.01	19.07
26.20	-26.40	157.00	53189.62	936.69	19.03
26.40	-26.60	116.15	52369.08	930.23	18.98
26.60	-26.80	86.02	51524.87	923.47	18.92
26.80	-27.00	64.01	50665.41	916.59	18.87
27.00	-27.20	48.07	49796.89	909.64	18.81
27.20	-27.40	36.56	48923.81	902.65	18.75
27.40	-27.60	28.28	48049.44	895.65	18.69
27.60	-27.80	22.33	47176.13	888.65	18.64
27.80	-28.00	18.06	46305.53	881.68	18.58
28.00	-28.20	14.98	45438.83	874.74	18.52
28.20	-28.40	12.76	44576.85	867.84	18.46
28.40	-28.60	11.17	43720.17	860.98	18.40
28.60	-28.80	10.01	12869.20	854.17	18.34
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18.28

29.20	-29.40	8.38	40353.19	834.02	18.17
29.40	-29.60	8.24	39527.41	827.41	18.11
29.60	-29.80	8.20	38708.20	820.85	18.05
29.80	-30.00	8.20	37895.54	814.35	18.00
30.00	-30.20	8.20	37089.40	807.89	17.94
30.20	-30.40	8.20	36289.71	801.49	17.88
30.40	-30.60	8.20	35496.42	795.14	17.82
30.60	-30.80	8.20	34709.48	788.84	17.77
30.80	-31.00	8.20	33928.84	782.59	17.71
31.00	-31.20	8.20	33154.46	774.24	17.64
31.20	-31.40	8.20	32388.42	764.76	17.55
31.40	-31.60	8.20	31631.87	755.40	17.46
31.60	-31.80	8.20	30884.67	746.15	17.38
31.80	-32.00	8.20	30146.72	737.02	17.29
32.00	-32.20	8.20	29417.91	728.00	17.21
32.20	-32.40	8.20	28698.11	719.10	17.12
32.40	-32.60	8.20	27987.21	710.30	17.04
32.60	-32.80	8.20	27285.11	701.61	16.95
32.80	-33.00	8.20	26591.70	693.03	16.87
33.00	-33.20	8.20	25906.87	684.56	16.79
33.20	-33.40	8.20	25230.52	676.19	16.71
33.40	-33.60	8.20	21562.53	667.92	16.62
33.60	-33.80	8.20	23902.80	659.76	16.54
33.80	-34.00	8.20	23251.24	651.70	16.46
34.00	-34.20	8.20	22607.75	543.74	16.38
34.20	-34.40	8.20	21972.21	635.87	16.30
34.40	-34.60	8.20	21344.54	628.11	16.22
34.60	-34.80	3.20	20724.63	620.44	16.14
34.80	-35.00	8.20	20112.40	612.86	16.06
35.00	-35.20	8.20	19507.74	605.38	15.98
35.20	-35.40	8.20	18910.56	597.99	15.91
35.40	-35.60	8.20	18320.77	590.69	15.33
35.60	-35.80	8.20	17738.28	583.49	15.75
35.80	-36.00	8.20	17163.00	576.37	15.67
36.00	-36.20	8.20	16594.83	567.09	15.57
36.20	-36.40	8.20	16035.95	557.91	15.47
36.40	-36.60	8.20	15486.24	548.88	15.37
36.60	-36.80	8.20	14945.56	540.00	15.27
36.80	-37.00	8.20	14413.75	531.27	15.17
37.00	-37.20	8.20	13890.69	522.68	15.07
37.20	-37.40	8.20	13376.21	514.23	14.97
37.40	-37.60	8.20	12870.19	505.92	14.83
37.60	-37.80	8.20	12372.47	497.74	14.78
37.80	-38.00	8.20	11882.93	489.70	14.68
38.00	-38.20	8.20	11401.43	481.79	14.59
38.20	-38.40	8.20	10927.84	474.02	14.49
38.40	-38.50	8.20	10462.02	466.37	14.40
38.60	-38.30	8.20	10003.86	458.84	14.31
38.80	-39.00	8.20	9553.22	444.87	14.13
39.00	-39.20	8.20	9116.55	427.67	13.91
39.20	-39.40	8.20	8697.08	411.15	13.69
39.40	-39.60	8.20	8294.13	395.27	13.48
39.60	-39.80	8.20	7907.06	379.04	13.25
39.80	-40.00	8.20	7536.23	362.99	13.03
40.00	-40.20	8.20	7181.44	347.64	12.80
40.20	-40.40	8.20	6842.00	332.96	12.58
40.40	-40.60	8.20	6517.24	318.91	12.37
40.60	-40.80	8.20	6206.54	305.47	12.16
40.80	-41.00	8.20	5909.27	292.60	11.95
41.00	-41.20	8.20	5624.87	280.30	11.75
41.20	-41.40	6.64	5351.21	268.46	11.55
41.40	-41.60	5.09	5087.84	257.07	11.35

TOTAL VOLUME EMERG SPLWY FLOW= 2041.08 AF

FLOOD ROUTING: 0.5 PMF EVENT

SINGLE 4' X 8' BOX CULVERTS

```
RESERVOIR ROUTING PROG.  (RES.BAS)  SMH.3-06-87

   WR GRACE DAM
   .5PMP
   LLB
   01-01-1992

 INPUT CONTROLS:
 NO OF STORAGE CURVE POINTS=  7 .  DELTA T=  .2

  CASE I  EMERG. SPLWY. CURVE. CREST LENGTH = 500 FT
 EARTH EMERG. SPILLWAY:  CREST EL.=  2900 . WIDTH=  8
      SIDE SLOPE=  .001 . EXIT SLOPE=  .04


  SPILLWAY DISCHARGE CURVES:
.............................
 CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
     500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED
                                   |     EMERG. SPLWY. VALUES       |
    ELEV      PRIN. Q      CHUTE Q        Q/FT    EFF. W.        TOT. Q     CTP
  2900.00       0.00         0.00         0.00      8.00           0.00     100
  2900.50       0.00         0.00         1.75      8.00          14.00     100
  2901.00       0.00         0.00         3.50      8.00          28.00     100
  2902.00       0.00         0.00         7.50      8.00          60.01     100
  2904.00       0.00         0.00        21.17      3.00         169.38     100
  2906.00       0.00         0.00        34.00      8.00         272.11     100
  2907.00       0.00         0.00        40.00      8.00         320.15     100
  2910.00       0.00         0.00        52.00      8.00         416.23     100
  2915.00       0.00         0.00        68.00      8.01         544.36     100
  2920.00       0.00         0.00        82.00      8.01         656.49     100
  2926.00       0.00         0.00        93.00      8.01         744.60     100
  **** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

  STORAGE INDICATION CURVE:
.............................
   ELEV.         STORAGE         S/^T+O/2     TOT. DIS.     CTR.
  2900.00          26.40          1597.20         0.00       100
  2900.50          30.19*         1833.33        14.00       100
  2901.00          34.52*         2102.29        28.00       100
  2902.00          45.13*         2760.12        60.01       100
  2904.00          77.11*         4749.58       169.38       100
  2906.00         131.70          8103.91       272.11       100
  2907.00         158.86*         9771.13       320.15       100
  2910.00         278.70         17069.46       416.23       100
  2915.00         549.70         33529.03       544.36       100
  2920.00         870.70         53005.59       656.49       100
  2926.00        1301.50         79113.05       744.60       100
   *--VALUE INSERTED BY LOG-LOG INTERP BY PROG.


 INITIAL ROUTING ELEV=  2900.00   STORAGE=       26.40
 ACTUAL DELTA T ROUTING INTERVAL=  .2  HRS..  PRINTOUT INTERVAL=  1  HRS.
 INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD55.PMP

 TIME INT..HRS      INFLOW. CFS         S/T+O/2   OUTFLOW. CFS   EXIT VEL
     INITIAL                            1597.20           0.00
   0.80    -1.00           7.02         1611.25           0.83       1.35
   1.80    -2.00           7.98         1642.41           2.68       2.15
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3.80	-4.00	8.00	1683.42	5.11	2.78
4.80	-5.00	8.00	1696.25	5.87	2.94
5.80	-6.00	8.00	1705.70	6.43	3.05
6.80	-7.00	10.35	1715.93	7.04	3.17
7.80	-8.00	59.39	1836.73	14.18	4.19
8.80	-9.00	209.31	2421.51	43.53	6.56
9.80	-10.00	597.13	4094.01	133.34	10.26
10.80	-11.00	3554.47	12844.70	360.61	15.28
11.80	-12.00	4678.04	34808.40	551.72	18.11
12.80	-13.00	2790.65	49244.00	634.83	19.16
13.80	-14.00	1931.21	57011.92	670.01	19.57
14.80	-15.00	1555.92	62092.45	687.16	19.77
15.80	-16.00	1324.07	65652.80	699.17	19.91
16.80	-17.00	1212.81	68400.82	708.45	20.01
17.80	-18.00	1100.68	70577.73	715.79	20.10
18.80	-19.00	1006.36	72200.94	721.27	20.16
19.80	-20.00	929.77	73359.84	725.18	20.20
20.80	-21.00	873.44	74230.32	728.12	20.23
21.80	-22.00	816.20	74757.77	729.90	20.25
22.80	-23.00	782.76	75098.97	731.05	20.27
23.80	-24.00	741.53	75218.61	731.46	20.27

PEAK
ELEV= 2925.11 STORAGE= 1237.37
TIME= 24.20 -24.40 INFLOW= 733.22 S/T 0/2= 75226.38
TOTAL SPLWY DIS= 731.48 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 731.48 EMRG EXIT VEL= 20.27

24.80	-25.00	679.79	75148.87	731.22	20.27
25.80	-26.00	282.04	73783.30	726.61	20.22
26.80	-27.00	64.01	70803.95	716.56	20.11
27.80	-28.00	18.06	67397.27	705.06	19.98
28.80	-29.00	9.20	63953.38	693.44	19.84
29.80	-30.00	8.20	60550.93	681.95	19.71
30.80	-31.00	8.20	57204.84	670.66	19.58
31.80	-32.00	8.20	53914.84	659.56	19.45
32.80	-33.00	8.20	50687.37	643.14	19.26
33.80	-34.00	8.20	47549.02	625.07	19.04
34.80	-35.00	8.20	44499.98	607.52	18.82
35.80	-36.00	8.20	41537.71	590.45	18.61
36.80	-37.00	8.20	38659.73	573.89	18.40
37.80	-38.00	8.20	35863.64	557.80	18.19
38.80	-39.00	8.20	33147.13	541.38	17.97
39.80	-40.00	8.20	30522.41	520.95	17.70
40.80	-41.00	8.20	27998.26	501.30	17.43

TOTAL VOLUME EMERG SPLWY FLOW= 1690.60 AF
TOTAL VOLUME OF HYD HOUTED= 2094.52 AF

FLOOD ROUTING: 0.55 PMF, NO EMERGENCY SPILLWAY

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH,3-06-87

WR GRACE DAM
FBD
LLB
12-14-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 , DELTA T= .2

CASE I EMERG. SPLWY. CURVE, CREST LENGTH = 500 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 , WIDTH= 12 '
SIDE SLOPE= .001 , EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW,2=HSO,3=BO,4=FB,10=LSW,20=LSO,100=ES,1000=CS
500=ES CURVE IS EXCEEDED, Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. Q	CHUTE Q	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.00	100
2901.00	0.00	0.00	3.50	12.00	42.00	100
2902.00	0.00	0.00	7.00	12.00	84.01	100
2904.00	0.00	0.00	20.33	12.00	244.05	100
2906.00	0.00	0.00	32.50	12.00	390.10	100
2907.00	0.00	0.00	38.00	12.00	456.14	100
2910.00	0.00	0.00	48.00	12.00	576.20	100
2915.00	0.00	0.00	65.00	12.01	780.33	100
2920.00	0.00	0.00	78.00	12.01	936.45	100
2926.00	0.00	0.00	90.00	12.01	1080.57	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:
....

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1597.20	0.00	100
2900.50	30.19*	1836.83	21.00	100
2901.00	34.52*	2109.29	42.00	100
2902.00	45.13*	2772.11	84.01	100
2904.00	77.11*	4786.91	244.05	100
2906.00	131.70	8162.90	390.10	100
2907.00	158,86*	9839.12	456.14	100
2910.00	278.70	17149.45	576.20	100
2915.00	549.70	33647.02	780.33	100
2920.00	870.70	53145.57	936.45	100
2926.00	1301.50	79281.03	1080.57	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS., PRINTOUT INTERVAL= 1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD5.PMP

TIME INT.,HRS		INFLOW, CFS	S/T+O/2	OUTFLOW, CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.80	-1.00	7.90	1612.89	1.37	1.40

3.80	-4.00	9.00	1677.63	7.05	2.69
4.80	-5.00	9.00	1685.82	7.77	2.80
5.80	-6.00	9.10	1691.12	8.23	2.86
6.80	-7.00	27.25	1726.99	11.37	3.26
7.80	-8.00	127.39	2030.68	35.94	5.17
8.80	-9.00	331.99	2953.57	98.42	7.73
9.80	-10.00	823.78	5092.35	257.26	11.35
10.80	-11.00	4434.61	15773.85	553.61	15.42
11.80	-12.00	5695.06	42049.00	847.60	18.29
12.80	-13.00	3341.43	58474.15	965.83	19.27
13.80	-14.00	2287.57	66664.52	1011.00	19.62
14.80	-15.00	1832.04	71542.82	1037.90	19.83
15.80	-16.00	1553.62	74566.58	1054.57	19.96
16.80	-17.00	1419.73	76608.12	1065.83	20.04
17.80	-18.00	1286.09	77967.94	1073.33	20.10
18.80	-19.00	1174.05	78678.52	1077.25	20.13
19.80	-20.00	1083.26	78853.98	1078.21	20.13

PEAK
ELEV= 2925.90 STORAGE= 1294.46
TIME= 19.80 -20.00 INFLOW= 1083.26 S/T O/2= 78853.98
TOTAL SPLWY DIS= 1078.21 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 1078.21 EMRG EXIT VEL= 20.13

20.80	-21.00	1016.47	78706.13	1077.40	20.13
21.80	-22.00	948.88	78176.46	1074.48	20.11
22.80	-23.00	909.18	77451.48	1070.48	20.08
23.80	-24.00	860.57	76492.83	1065.19	20.04
24.80	-25.00	788.35	75340.38	1058.84	19.99
25.80	-26.00	326.83	72717.78	1044.38	19.88
26.80	-27.00	73.96	68277.61	1019.89	19.69
27.80	-28.00	20.66	63408.07	993.04	19.48
28.80	-29.00	10.39	58562.54	966.32	19.27
29.80	-30.00	9.23	53830.38	940.22	19.06
30.80	-31.00	9.23	49242.63	905.20	18.77
31.80	-32.00	9.23	44833.97	869.90	18.48
32.80	-33.00	9.23	40599.00	835.99	18.19
33.80	-34.00	9.23	36530.88	803.42	17.90
34.80	-35.00	9.23	32624.14	767.67	17.58
35.80	-36.00	9.23	28924.63	721.90	17.15
36.80	-37.00	9.23	25448.40	678.89	16.73
37.80	-38.00	9.23	22181.99	638.47	16.33
38.80	-39.00	9.23	19112.71	600.49	15.93
39.80	-40.00	9.23	16230.13	561.10	15.51
40.80	-41.00	9.23	13559.96	517.25	15.01

TOTAL VOLUME EMERG SPLWY FLOW= 2356.07 AF
TOTAL VOLUME OF HYD ROUTED= 2520.77 AF

FLOOD ROUTING: 0.66 PMF W/ EMERGENCY SPILLWAY

TWIN 4' X 6' BOX CULVERTS

ESERVOIR ROUTING PROG. (RES.BAS) SMH.3-06-87

WR GRACE DAM
.66PMP
LLB
12-14-1991

INPUT CONTROLS:
NO OF STORAGE CURVE POINTS= 7 . DELTA T= .2

CASE I EMERG. SPLWY. CURVE. CREST LENGTH = 100 FT
EARTH EMERG. SPILLWAY: CREST EL.= 2900 . WIDTH= 12
SIDE SLOPE= .01 . EXIT SLOPE= .04

SPILLWAY DISCHARGE CURVES:
....
CTR: 1=HSW.2=HSO.3=BO.4=FB.10=LSW.20=LSO.100=ES.1000=CS
500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED

			EMERG. SPLWY. VALUES			
ELEV	PRIN. O	CHUTE O	Q/FT	EFF. W.	TOT. Q	CTR
2900.00	0.00	0.00	0.00	12.00	0.00	100
2900.50	0.00	0.00	1.75	12.00	21.01	100
2901.00	0.00	0.00	3.50	12.01	42.03	100
2902.00	0.00	0.00	7.00	12.01	84.08	100
2904.00	0.00	0.00	20.33	12.02	244.48	100
2906.00	0.00	0.00	32.50	12.03	391.04	100
2907.00	0.00	0.00	38.00	12.04	457.35	100
2910.00	0.00	0.00	48.00	12.04	577.99	100
2915.00	0.00	0.00	65.00	12.05	783.30	100
2920.00	0.00	0.00	78.00	12.06	940.48	100
2926.00	0.00	0.00	173.30	12.10	2096.54	100

**** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****

STORAGE INDICATION CURVE:
....

ELEV.	STORAGE	S/^T+O/2	TOT. DIS.	CTR.
2900.00	26.40	1597.20	0.00	100
2900.50	30.19*	1836.84	21.01	100
2901.00	34.52*	2109.30	42.03	100
2902.00	45.13*	2772.15	84.08	100
2904.00	77.11*	4787.12	244.48	100
2906.00	131.70	8163.37	391.04	100
2907.00	158.86*	9839.73	457.35	100
2910.00	278.70	17150.35	577.99	100
2915.00	549.70	33648.50	783.30	100
2920.00	870.70	53147.59	940.48	100
2926.00	1301.50	79789.02	2096.54	100

*--VALUE INSERTED BY LOG-LOG INTERP BY PROG.

INITIAL ROUTING ELEV= 2900.00 STORAGE= 26.40
ACTUAL DELTA T ROUTING INTERVAL= .2 HRS.. PRINTOUT INTERVAL= .1 HRS.
INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: HYD75.PMP

TIME INT..HRS		INFLOW, CFS	S/T+O/2	OUTFLOW, CFS	EXIT VEL
INITIAL			1597.20	0.00	
0.40	-0.60	3.89	1599.28	0.18	0.62
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

1.00	-1.20	8.49	1620.00	2.06	1.63
1.20	-1.40	8.76	1626.77	2.59	1.80
1.40	-1.60	8.89	1633.07	3.14	1.95
1.60	-1.80	8.95	1638.87	3.65	2.07
1.80	-2.00	8.98	1644.19	4.12	2.17
2.00	-2.20	8.99	1649.06	4.55	2.26
2.20	-2.40	8.99	1653.51	4.94	2.33
2.40	-2.60	9.00	1657.57	5.29	2.40
2.60	-2.80	9.00	1661.28	5.62	2.46
2.80	-3.00	9.00	1664.66	5.91	2.51
3.00	-3.20	9.00	1667.75	6.18	2.56
3.20	-3.40	9.00	1670.56	6.43	2.60
3.40	-3.60	9.00	1673.13	6.66	2.63
3.60	-3.80	9.00	1675.47	6.86	2.56
3.80	-4.00	9.00	1677.61	7.05	2.69
4.00	-4.20	9.00	1679.55	7.22	2.72
4.20	-4.40	9.00	1681.34	7.38	2.74
4.40	-4.60	9.00	1682.97	7.52	2.76
4.60	-4.80	9.00	1684.45	7.65	2.78
4.80	-5.00	9.01	1685.80	7.77	2.80
5.00	-5.20	9.04	1687.08	7.88	2.82
5.20	-5.40	9.18	1683.33	7.99	2.83
5.40	-5.60	9.63	1690.01	8.14	2.85
5.60	-5.80	10.89	1692.77	8.38	2.89
5.80	-6.00	13.87	1698.26	8.86	2.95
6.00	-6.20	19.71	1709.11	9.81	3.07
6.20	-6.40	29.40	1728.70	11.53	3.28
6.40	-6.60	43.39	1760.56	14.32	3.57
6.60	-6.80	61.69	1807.93	18.47	3.96
6.80	-7.00	83.93	1873.39	23.83	4.38
7.00	-7.20	109.61	1959.17	30.44	4.83
7.20	-7.40	138.44	2067.17	33.78	5.32
7.40	-7.60	170.41	2198.80	47.70	5.78
7.60	-7.80	205.78	2356.88	57.73	6.24
7.80	-8.00	244.94	2544.09	69.61	6.73
8.00	-8.20	288.29	2762.77	83.49	7.23
3.20	-8.40	336.09	3015.37	103.44	7.88
8.40	-8.60	388.47	3300.40	126.13	8.53
8.60	-8.80	446.26	3620.53	151.61	9.18
8.80	-9.00	511.57	3980.49	180.27	9.84
9.00	-9.20	588.27	4388.48	212.74	10.51
9.20	-9.40	681.31	4857.05	247.51	11.17
9.40	-9.60	796.21	5405.75	271.33	11.59
9.60	-9.80	940.31	6074.74	300.37	12.07
9.80	-10.00	1131.76	6906.12	336.46	12.63
10.00	-10.20	1430.46	8000.12	383.95	13.31
10.20	-10.40	1959.12	9575.28	446.89	14.14
10.40	-10.60	2850.23	11978.63	492.65	14.70
10.60	-10.80	4108.81	15594.78	552.32	15.39
10.80	-11.00	5520.77	20563.23	620.47	16.12
11.00	-11.20	6742.67	26685.44	696.65	16.88
11.20	-11.40	7504.32	33493.10	781.37	17.67
11.40	-11.60	7719.71	40431.44	837.98	18.17
11.60	-11.80	7462.33	47055.80	891.37	18.63
11.80	-12.00	6902.97	53067.39	939.83	19.02
12.00	-12.20	6226.64	58354.21	1166.41	20.74
12.20	-12.40	5556.76	62744.56	1356.92	22.02
12.40	-12.60	4948.50	66336.14	1512.77	23.00
12.60	-12.80	4420.90	69244.27	1638.97	23.74
12.80	-13.00	3975.42	71580.72	1740.35	24.32
13.00	-13.20	3604.71	73445.08	1821.25	24.76
13.20	-13.40	3299.09	74922.91	1385.38	25.11
13.40	-13.60	3050.34	76087.87	1935.93	25.37
13.60	-13.80	2850.06	77001.99	1975.60	25.58

14.20	-14.40	2437.31	78669.18	2047.95	25.95
14.40	-14.60	2330.93	78952.16	2060.23	26.01
14.60	-14.80	2231.27	79123.20	2067.65	26.04
14.80	-15.00	2138.20	79193.75	2070.71	26.06

PEAK

ELEV= 2925.87 STORAGE= 1291.87
TIME= 14.80 -15.00 INFLOW= 2138.20 S/T 0/2= 79193.75
TOTAL SPLWY DIS= 2070.71 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 2070.71 EMRG EXIT VEL= 26.06

15.00	-15.20	2053.33	79176.37	2069.95	26.06
15.20	-15.40	1977.85	79084.26	2065.96	26.04
15.40	-15.60	1911.89	78930.19	2059.27	26.00
15.60	-15.80	1854.78	78725.70	2050.40	25.96
15.80	-16.00	1805.97	78481.27	2039.79	25.90
16.00	-16.20	1765.03	78206.51	2027.87	25.84
16.20	-16.40	1730.77	77909.41	2014.98	25.78
16.40	-16.60	1701.07	77595.50	2001.36	25.71
16.60	-16.80	1673.48	77267.63	1987.13	25.64
16.80	-17.00	1645.91	76926.41	1972.32	25.56
17.00	-17.20	1616.91	76571.01	1956.90	25.43
17.20	-17.40	1585.88	76199.99	1940.80	25.40
17.40	-17.60	1553.21	75312.40	1923.98	25.31
17.60	-17.80	1519.99	75408.41	1906.45	25.22
17.80	-18.00	1487.82	74989.78	1888.28	25.12
18.00	-18.20	1458.14	74559.64	1869.62	25.02
18.20	-18.40	1431.32	74121.34	1850.60	24.92
18.40	-18.60	1406.37	73677.12	1831.32	24.82
18.60	-18.80	1381.63	73227.43	1811.81	24.71
18.80	-19.00	1355.76	72771.38	1792.02	24.60
19.00	-19.20	1328.60	72307.95	1771.91	24.49
19.20	-19.40	1301.93	71837.97	1751.52	24.38
19.40	-19.60	1278.85	71365.30	1731.01	24.27
19.60	-19.80	1261.43	70895.72	1710.63	24.15
19.80	-20.00	1248.97	70434.06	1690.60	24.04
20.00	-20.20	1238.56	69982.03	1670.98	23.93
20.20	-20.40	1226.83	69537.88	1651.71	23.82
20.40	-20.60	1211.51	69097.68	1632.61	23.71
20.60	-20.80	1192.18	68657.26	1513.49	23.60
20.80	-21.00	1170.41	68214.17	1594.27	23.48
21.00	-21.20	1148.72	67768.63	1574.93	23.37
21.20	-21.40	1129.24	67322.93	1555.59	23.26
21.40	-21.60	1113.08	66880.42	1536.39	23.14
21.60	-21.80	1100.57	66444.60	1517.48	23.03
21.80	-22.00	1091.27	66018.39	1498.99	22.92
22.00	-22.20	1084.06	65603.47	1480.98	22.81
22.20	-22.40	1077.25	65199.74	1463.46	22.70
22.40	-22.60	1068.94	64805.22	1446.34	22.59
22.60	-22.80	1057.94	64416.81	1429.49	22.49
22.80	-23.00	1044.51	64031.84	1412.78	22.38
23.00	-23.20	1030.07	63649.13	1396.17	22.28
23.20	-23.40	1016.24	63269.19	1379.69	22.17
23.40	-23.60	1004.30	62893.80	1363.40	22.07
23.60	-23.80	994.85	62525.25	1347.41	21.96
23.80	-24.00	987.71	62165.55	1331.80	21.86
24.00	-24.20	981.98	61815.74	1316.62	21.76
24.20	-24.40	975.99	61475.11	1301.84	21.66
24.40	-24.60	966.25	61139.53	1287.28	21.57
24.60	-24.80	945.68	60797.93	1272.45	21.47
24.80	-25.00	904.01	60429.49	1256.46	21.38
25.00	-25.20	832.32	60005.34	1238.06	21.23
25.20	-25.40	730.63	59497.91	1216.04	21.08
25.40	-25.60	610.20	58892.08	1189.75	20.90
25.60	-25.80	486.77	58189.10	1159.25	20.68

26.00	-26.20	280.09	56558.67	1088.50	20.17
26.20	-26.40	207.39	55677.57	1050.26	19.89
26.40	-26.60	153.00	54780.21	1011.33	19.59
26.60	-26.80	112.88	53881.86	972.34	19.28
26.80	-27.00	83.58	52993.10	939.23	19.02
27.00	-27.20	62.34	52116.21	932.16	18.96
27.20	-27.40	47.02	51231.07	925.03	18.90
27.40	-27.60	36.00	50342.04	917.86	18.85
27.60	-27.80	28.08	49452.25	910.69	18.79
27.80	-28.00	22.39	48563.95	903.53	18.73
28.00	-28.20	18.29	47678.71	896.40	18.67
28.20	-28.40	15.34	46797.66	889.29	18.61
28.40	-28.60	13.22	45921.58	882.23	18.55
28.60	-28.80	11.68	45051.03	875.22	18.49
28.80	-29.00	10.60	44186.41	868.25	13.43
29.00	-29.20	9.89	43328.06	861.33	18.37
29.20	-29.40	9.50	42476.23	854.46	18.32
29.40	-29.60	9.32	41631.09	847.65	18.26
29.60	-29.80	9.27	40792.71	840.89	18.20
29.80	-30.00	9.27	39961.09	834.19	13.14
30.00	-30.20	9.27	39136.17	827.54	18.08
30.20	-30.40	9.27	38317.90	820.94	18.03
30.40	-30.60	9.27	37506.23	814.40	17.97
30.60	-30.80	9.27	36701.09	807.91	17.91
30.80	-31.00	9.27	35902.45	801.47	17.85
31.00	-31.20	9.27	35110.25	795.09	17.80
31.20	-31.40	9.27	34324.43	788.75	17.74
31.40	-31.60	9.27	33544.95	782.02	17.68
31.60	-31.80	9.27	32772.20	772.40	17.59
31.80	-32.00	9.27	32009.07	762.90	17.51
32.00	-32.20	9.27	31255.43	753.52	17.42
32.20	-32.40	9.27	30511.13	744.26	17.33
32.40	-32.60	9.27	29776.18	735.12	17.25
32.60	-32.80	9.27	29050.34	726.08	17.16
32.80	-33.00	9.27	28333.52	717.16	17.08
33.00	-33.20	9.27	27625.63	708.35	17.00
33.20	-33.40	9.27	26926.54	699.65	16.91
33.40	-33.60	9.27	26236.16	691.06	16.83
33.60	-33.80	9.27	25554.37	682.58	16.75
33.80	-34.00	9.27	24881.06	674.20	16.66
34.00	-34.20	9.27	24216.13	665.92	16.58
34.20	-34.40	9.27	23559.47	657.75	16.50
34.40	-34.60	9.27	22910.99	649.68	16.42
34.60	-34.80	9.27	22270.58	641.71	16.34
34.80	-35.00	9.27	21638.13	633.84	16.26
35.00	-35.20	9.27	21013.56	626.07	16.18
35.20	-35.40	9.27	20396.76	618.39	16.10
35.40	-35.60	9.27	19787.63	610.81	16.02
35.60	-35.80	9.27	19186.09	603.33	15.94
35.80	-36.00	9.27	18592.03	595.93	15.86
36.00	-36.20	9.27	18005.36	588.63	15.79
36.20	-36.40	9.27	17426.00	581.42	15.71
36.40	-36.60	9.27	16853.84	573.10	15.62
36.60	-36.80	9.27	16290.01	563.80	15.52
36.80	-37.00	9.27	15735.48	554.64	15.42
37.00	-37.20	9.27	15190.11	545.64	15.31
37.20	-37.40	9.27	14653.73	536.79	15.22
37.40	-37.60	9.27	14126.21	528.09	15.12
37.60	-37.80	9.27	13607.39	519.53	15.02
37.80	-38.00	9.27	13097.13	511.11	14.92
38.00	-38.20	9.27	12595.29	502.82	14.82
38.20	-38.40	9.27	12101.74	494.68	14.73
38.40	-38.60	9.27	11616.33	486.67	14.63
38.60	-38.80	9.27	11138.92	478.79	14.54
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

39.20	-39.40	9.27	9753.48	453.94	14.23
39.40	-39.60	9.27	9308.81	436.35	14.01
39.60	-39.80	9.27	8831.72	419.46	13.79
39.80	-40.00	9.27	8471.54	403.23	13.57
40.00	-40.20	9.27	8077.58	387.32	13.36
40.20	-40.40	9.27	7699.53	370.91	13.13
40.40	-40.60	9.27	7337.89	355.21	12.90
40.60	-40.80	9.27	6991.95	340.19	12.68
40.80	-41.00	9.27	6661.03	325.82	12.47
41.00	-41.20	9.26	6344.47	312.08	12.25

TOTAL VOLUME EMERG SPLWY FLOW= 2947.67 AF
TOTAL VOLUME OF HYD ROUTED= 3020.96 AF

FLOOD ROUTING: 0.66 PMF W/ EMERGENCY SPILLWAY

SINGLE 5' X 9' BOX CULVERT

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RESERVOIR ROUTING PROG.  (RES.BAS)  SMH.3-06-87

   WR GRACE DAM
   .75PMP
   LLB
   03-19-1992

 INPUT CONTROLS:
 NO OF STORAGE CURVE POINTS=  7 .  DELTA T=  .2

  CASE I  EMERG. SPLWY. CURVE. CREST LENGTH = 500 FT
 EARTH EMERG. SPILLWAY:  CREST EL.=  2900 , WIDTH=  9 '
       SIDE SLOPE=  .001 . EXIT SLOPE=  .04


  SPILLWAY DISCHARGE CURVES:
............................
 CTR: 1=HSW.2=HSO.3=BO.4=FB,10=LSW.20=LSO.100=ES.1000=CS
      500=ES CURVE IS EXCEEDED. Q'S ARE EXTRAPOLATED
                                   !      EMERG. SPLWY. VALUES      !
    ELEV      PRIN. Q       CHUTE Q        Q/FT    EFF. W.          TOT. Q      CTR
  2900.00       0.00          0.00         0.00       9.00            0.00      100
  2900.50       0.00          0.00         1.75       9.00           15.75      100
  2901.00       0.00          0.00         3.50       9.00           31.50      100
  2902.00       0.00          0.00         7.50       9.00           67.51      100
  2904.00       0.00          0.00        21.17       9.00          190.55      100
  2906.00       0.00          0.00        36.50       9.00          328.63      100
  2907.00       0.00          0.00        45.00       9.00          405.18      100
  2910.00       0.00          0.00        62.00       9.00          558.31      100
  2915.00       0.00          0.00        90.00       9.01          810.57      100
  2920.00       0.00          0.00       100.00       9.01          900.68      100
  2926.00       0.00          0.00       195.00       9.01         1757.06      100
  **** WARNING: DELTA T MAY BE TOO LARGE FOR PROPER ROUTING ****


 INITIAL ROUTING ELEV=  2900.00   STORAGE=      26.40
 ACTUAL DELTA T ROUTING INTERVAL=  .2  HRS..  PRINTOUT INTERVAL=  1  HRS.
 INFLOW Q INTEGRATED FROM TIME-Q DATA IN FILE: RYD75.PMP

 TIME INT.,HRS      INFLOW. CFS        S/T+0/2   OUTFLOW, CFS   EXIT VEL
      INITIAL                          1597.20         0.00
    0.80    -1.00         7.90         1613.11         1.06         1.41
    1.80    -2.00         8.98         1647.14         3.32         2.24
    2.80    -3.00         9.00         1671.99         4.97         2.63
    3.80    -4.00         9.00         1689.64         6.14         2.86
    4.80    -5.00         9.01         1702.15         6.97         3.01
    5.80    -6.00        13.87         1718.38         8.05         3.19
    6.80    -7.00        83.93         1901.35        19.67         4.55
    7.80    -8.00       244.94         2607.06        58.95         7.06
    8.80    -9.00       511.57         4124.10       151.35        10.30
    9.80   -10.00      1131.76         7245.58       292.32        13.40
   10.80   -11.00      5520.77        21076.87       618.41        18.08
   11.80   -12.00      6902.97        53547.65       914.25        21.14
   12.80   -13.00      3975.42        72644.48      1531.59        25.99
   13.80   -14.00      2688.27        79918.49      1766.73        27.51
   14.80   -15.00      2138.20        82565.72      1852.31        28.04

     **PEAK**
```

TOTAL SPLWY DIS= 1864.25 CTR= 100
PRIN Q= 0.00 CHUTE Q= 0.00
EMRG Q= 1864.25 EMRG EXIT VEL= 28.11

15.80	-16.00	1805.97	82867.57	1862.07	28.10
16.80	-17.00	1615.91	82114.12	1837.71	27.95
17.80	-18.00	1487.82	80768.58	1794.21	27.68
18.80	-19.00	1355.76	78944.20	1735.24	27.32
19.80	-20.00	1248.97	76822.88	1666.66	26.88
20.80	-21.00	1170.41	74667.37	1596.98	26.43
21.80	-22.00	1091.27	72411.28	1524.05	25.94
22.80	-23.00	1044.51	70263.88	1454.63	25.46
23.80	-24.00	987.71	68160.83	1386.65	24.98
24.80	-25.00	904.01	66130.89	1321.03	24.50
25.80	-26.00	373.90	62747.38	1211.65	23.66
26.80	-27.00	83.58	57837.95	1052.94	22.37
27.80	-28.00	22.39	53083.33	900.47	21.02
28.80	-29.00	10.60	48690.87	880.14	20.83
29.80	-30.00	9.27	44377.54	860.17	20.63
30.80	-31.00	9.27	40162.23	840.66	20.45
31.80	-32.00	9.27	36043.58	821.59	20.26
32.80	-33.00	9.27	32028.85	785.63	19.90
33.80	-34.00	9.27	28263.79	728.14	19.31
34.80	-35.00	9.27	24777.51	674.91	18.73
35.80	-36.00	9.27	21549.33	625.62	18.17
36.80	-37.00	9.27	18560.29	579.98	17.63
37.80	-38.00	9.27	15798.58	530.26	17.01
38.80	-39.00	9.27	13300.25	478.05	16.32
39.80	-40.00	9.27	11052.30	431.07	15.65
40.80	-41.00	9.27	9039.42	369.93	14.73

TOTAL VOLUME EMERG SPLWY FLOW= 2909.77 AF
TOTAL VOLUME OF HYD ROUTED= 3020.96 AF

INLET CHANNEL HYDRAULICS

COOPERATOR OR PROJECT= WR GRACE

DESCRIPTION= INLET CHANNEL

CALCULATED BY LLB

12-14-1991

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
1.0	12.00	10.0	2.0 TO 1	.00300	.0350	2.05	24.60
1.1	13.42	10.0	2.0 TO 1	.00300	.0350	2.17	29.12
1.2	14.88	10.0	2.0 TO 1	.00300	.0350	2.28	33.93
1.3	16.38	10.0	2.0 TO 1	.00300	.0350	2.39	39.15
1.4	17.92	10.0	2.0 TO 1	.00300	.0350	2.48	44.44
1.5	19.50	10.0	2.0 TO 1	.00300	.0350	2.58	50.31
1.6	21.12	10.0	2.0 TO 1	.00300	.0350	2.67	56.39
1.7	22.78	10.0	2.0 TO 1	.00300	.0350	2.76	62.87
1.8	24.48	10.0	2.0 TO 1	.00300	.0350	2.85	59.77
1.9	26.22	10.0	2.0 TO 1	.00300	.0350	2.94	77.09
2.0	28.00	10.0	2.0 TO 1	.00300	.0350	3.02	84.56
2.1	29.82	10.0	2.0 TO 1	.00300	.0350	3.10	92.44
2.2	31.68	10.0	2.0 TO 1	.00300	.0350	3.18	100.74
2.3	33.58	10.0	2.0 TO 1	.00300	.0350	3.26	109.47
2.4	35.52	10.0	2.0 TO 1	.00300	.0350	3.33	118.28
2.5	37.50	10.0	2.0 TO 1	.00300	.0350	3.40	127.50
2.6	39.52	10.0	2.0 TO 1	.00300	.0350	3.48	137.53
2.7	41.58	10.0	2.0 TO 1	.00300	.0350	3.54	147.19
2.8	43.68	10.0	2.0 TO 1	.00300	.0350	3.62	158.12
2.9	45.82	10.0	2.0 TO 1	.00300	.0350	3.68	168.62
3.0	48.00	10.0	2.0 TO 1	.00300	.0350	3.75	180.00
3.1	50.22	10.0	2.0 TO 1	.00300	.0350	3.81	191.34
3.2	52.48	10.0	2.0 TO 1	.00300	.0350	3.89	204.15
3.3	54.78	10.0	2.0 TO 1	.00300	.0350	[illegible]	[illegible]

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
3.4	57.12	10.0	2.0 TO 1	.00300	.0350	4.02	229.52
3.5	59.50	10.0	2.0 TO 1	.00300	.0350	4.08	242.76
3.6	61.92	10.0	2.0 TO 1	.00300	.0350	4.13	255.73
3.7	64.38	10.0	2.0 TO 1	.00300	.0350	4.19	269.75
3.8	66.88	10.0	2.0 TO 1	.00300	.0350	4.26	284.91
3.9	69.42	10.0	2.0 TO 1	.00300	.0350	4.32	299.89
4.0	72.00	10.0	2.0 TO 1	.00300	.0350	4.38	315.36
4.1	74.62	10.0	2.0 TO 1	.00300	.0350	4.43	330.57
4.2	77.28	10.0	2.0 TO 1	.00300	.0350	4.50	347.76
4.3	79.98	10.0	2.0 TO 1	.00300	.0350	4.56	364.71
4.4	82.72	10.0	2.0 TO 1	.00300	.0350	4.61	381.34
4.5	85.50	10.0	2.0 TO 1	.00300	.0350	4.67	399.29
4.6	88.32	10.0	2.0 TO 1	.00300	.0350	4.72	416.87
4.7	91.18	10.0	2.0 TO 1	.00300	.0350	4.77	434.93
4.8	94.08	10.0	2.0 TO 1	.00300	.0350	4.83	454.41
4.9	97.02	10.0	2.0 TO 1	.00300	.0350	4.88	473.46
5.0	100.00	10.0	2.0 TO 1	.00300	.0350	4.94	494.00
5.1	103.02	10.0	2.0 TO 1	.00300	.0350	4.99	514.07
5.2	106.08	10.0	2.0 TO 1	.00300	.0350	5.04	534.64
5.3	109.18	10.0	2.0 TO 1	.00300	.0350	5.09	555.73
5.4	112.32	10.0	2.0 TO 1	.00300	.0350	5.15	578.45
5.5	115.50	10.0	2.0 TO 1	.00300	.0350	5.20	600.60
5.6	118.72	10.0	2.0 TO 1	.00300	.0350	5.25	623.28
5.7	121.98	10.0	2.0 TO 1	.00300	.0350	5.30	646.49

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
5.8	125.28	10.0	2.0 TO 1	.00300	.0350	5.35	670.25
5.9	128.62	10.0	2.0 TO 1	.00300	.0350	5.39	693.26
6.0	132.00	10.0	2.0 TO 1	.00300	.0350	5.44	718.08
6.1	135.42	10.0	2.0 TO 1	.00300	.0350	5.50	744.81
6.2	138.88	10.0	2.0 TO 1	.00300	.0350	5.55	770.78
6.3	142.38	10.0	2.0 TO 1	.00300	.0350	5.60	797.33
6.4	145.92	10.0	2.0 TO 1	.00300	.0350	5.65	824.45
6.5	149.50	10.0	2.0 TO 1	.00300	.0350	5.70	852.15
6.6	153.12	10.0	2.0 TO 1	.00300	.0350	5.73	877.38
6.7	156.78	10.0	2.0 TO 1	.00300	.0350	5.78	906.19
6.8	160.48	10.0	2.0 TO 1	.00300	.0350	5.83	935.60
6.9	164.22	10.0	2.0 TO 1	.00300	.0350	5.88	965.51
7.0	168.00	10.0	2.0 TO 1	.00300	.0350	5.93	996.24

OUTLET CHANNEL HYDRAULICS

COOPERATOR OR PROJECT= WR GRACE DAM

DESCRIPTION= OUTLET CHANNEL

CALCULATED BY LLB

12-13-1991

DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
1.0	11.50	10.0	1.5 TO 1	.04000	.0400	6.61	76.01
1.1	12.82	10.0	1.5 TO 1	.04000	.0400	7.03	90.12
1.2	14.16	10.0	1.5 TO 1	.04000	.0400	7.38	104.50
1.3	15.54	10.0	1.5 TO 1	.04000	.0400	7.72	119.97
1.4	16.94	10.0	1.5 TO 1	.04000	.0400	3.06	136.54
1.5	18.38	10.0	1.5 TO 1	.04000	.0400	8.34	153.29
1.6	19.84	10.0	1.5 TO 1	.04000	.0400	8.67	172.01
1.7	21.34	10.0	1.5 TO 1	.04000	.0400	8.94	190.78
1.8	22.86	10.0	1.5 TO 1	.04000	.0400	9.26	211.68
1.9	24.42	10.0	1.5 TO 1	.04000	.0400	9.52	232.48
2.0	26.00	10.0	1.5 TO 1	.04000	.0400	9.78	254.28
2.1	27.62	10.0	1.5 TO 1	.04000	.0400	10.04	277.30
2.2	29.26	10.0	1.5 TO 1	.04000	.0400	10.29	301.09
2.3	30.93	10.0	1.5 TO 1	.04000	.0400	10.54	326.00
2.4	32.64	10.0	1.5 TO 1	.04000	.0400	10.79	352.19
2.5	34.37	10.0	1.5 TO 1	.04000	.0400	11.04	379.44
2.6	36.14	10.0	1.5 TO 1	.04000	.0400	11.28	407.66
2.7	37.93	10.0	1.5 TO 1	.04000	.0400	11.48	435.44
2.8	39.76	10.0	1.5 TO 1	.04000	.0400	11.72	465.99
2.9	41.61	10.0	1.5 TO 1	.04000	.0400	11.91	495.58

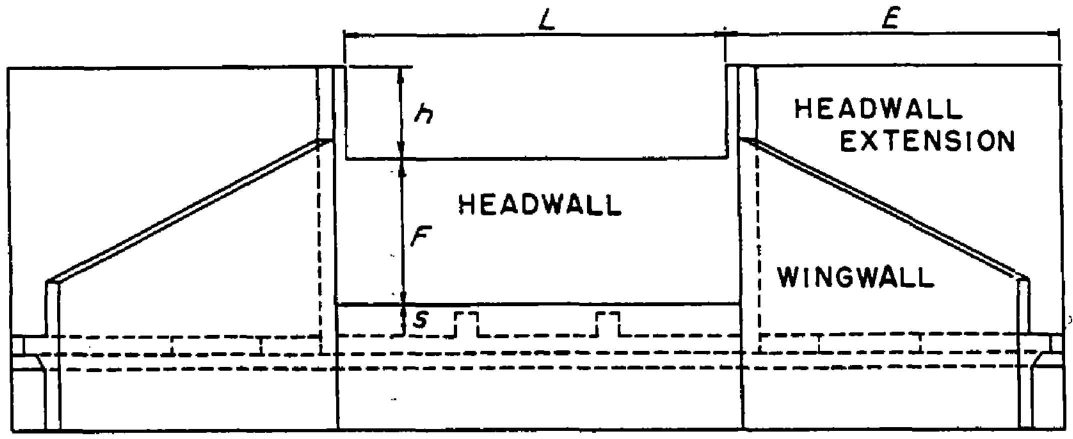
DITCH HYDRAULICS PROGRAM

DEPTH FT.	AREA SQ.FT.	B.W. FT.	AVE. S.S.	CHAN. GRADE	MAN. N	VEL. F.P.S.	FLOW CFS
3.0	43.50	10.0	1.5 TO 1	.04000	.0400	12.15	528.52
3.1	45.41	10.0	1.5 TO 1	.04000	.0400	12.34	560.36
3.2	47.36	10.0	1.5 TO 1	.04000	.0400	12.57	595.32
3.3	49.33	10.0	1.5 TO 1	.04000	.0400	12.76	629.45
3.4	51.34	10.0	1.5 TO 1	.04000	.0400	12.99	666.91
3.5	53.37	10.0	1.5 TO 1	.04000	.0400	13.17	702.88
3.6	55.44	10.0	1.5 TO 1	.04000	.0400	13.36	740.63
3.7	57.53	10.0	1.5 TO 1	.04000	.0400	13.54	778.96
3.8	59.66	10.0	1.5 TO 1	.04000	.0400	13.76	820.92
3.9	61.81	10.0	1.5 TO 1	.04000	.0400	13.94	861.63
4.0	64.00	10.0	1.5 TO 1	.04000	.0400	14.13	904.32
4.1	66.21	10.0	1.5 TO 1	.04000	.0400	14.30	946.80
4.2	68.45	10.0	1.5 TO 1	.04000	.0400	14.48	991.30
4.3	70.73	10.0	1.5 TO 1	.04000	.0400	14.66	1036.90
4.4	73.04	10.0	1.5 TO 1	.04000	.0400	14.84	1083.91
4.5	75.37	10.0	1.5 TO 1	.04000	.0400	15.01	1131.30
4.6	77.74	10.0	1.5 TO 1	.04000	.0400	15.18	1180.09
4.7	80.13	10.0	1.5 TO 1	.04000	.0400	15.36	1230.80
4.8	82.56	10.0	1.5 TO 1	.04000	.0400	15.53	1282.16
4.9	85.01	10.0	1.5 TO 1	.04000	.0400	15.70	1334.66
5.0	87.50	10.0	1.5 TO 1	.04000	.0400	15.87	1338.63

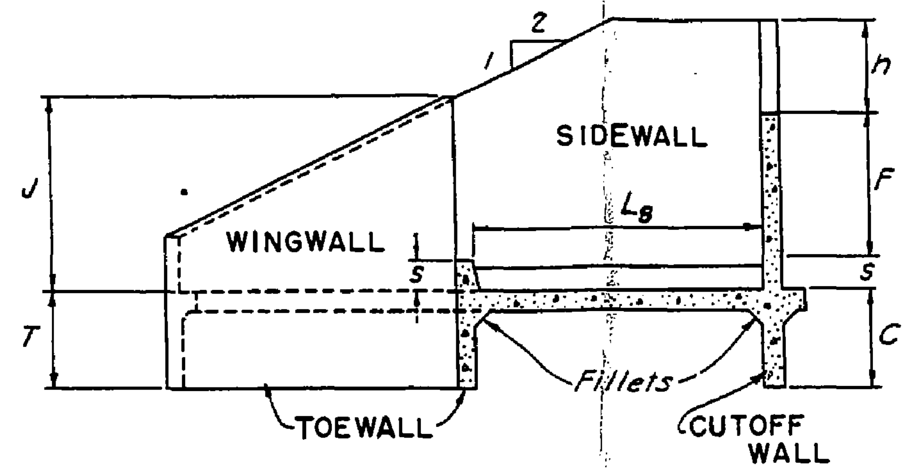
APPENDIX E

STANDARD DRAWING - SCS DROP STRUCTURE

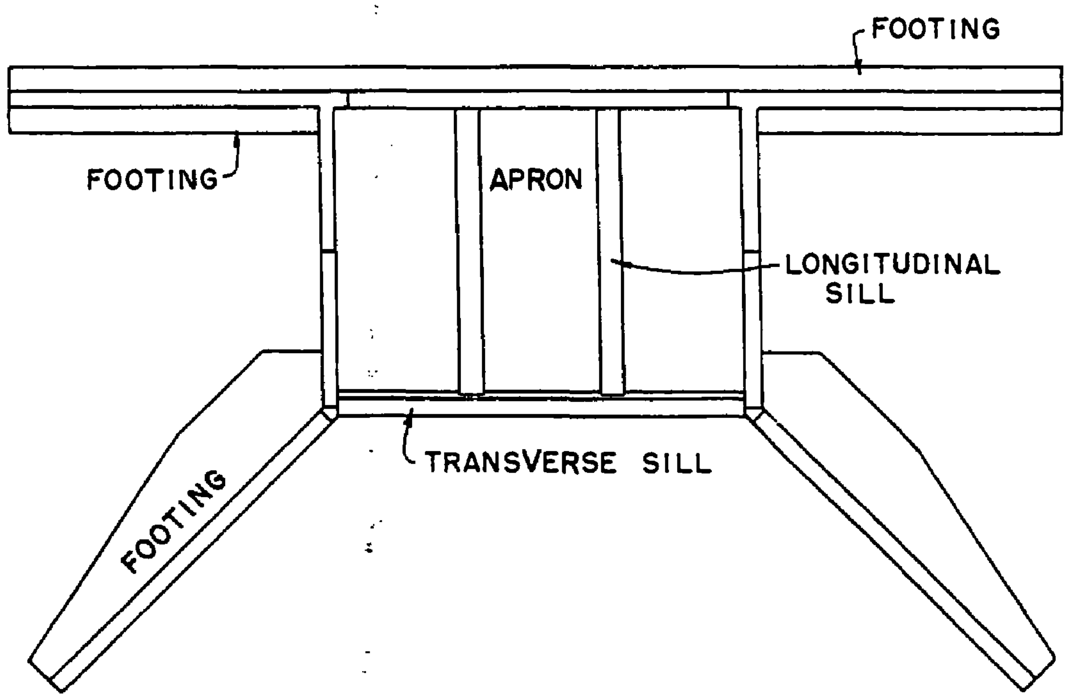
DROP SPILLWAYS: NOMENCLATURE AND SYMBOLS OF DROP SPILLWAY



DOWNSTREAM ELEVATION



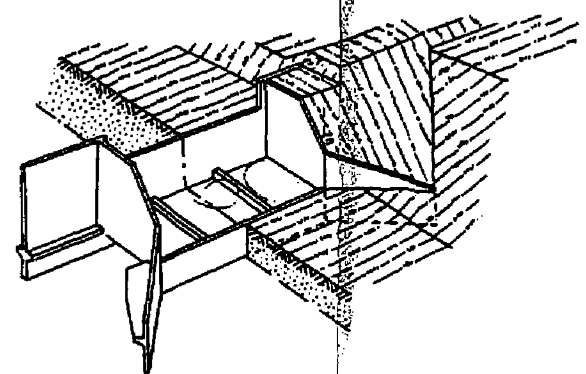
SECTION ON CENTER LINE



PLAN

SYMBOLS

- L = Length of weir.
- h = Depth of weir.
- F = Drop through spillway from crest of weir to top of transverse sill.
- s = Height of transverse sill.
- L_B = Length of apron.
- T = Depth of toewall below top of apron.
- C = Depth of cutoff wall below top of apron.
- d_C = Criticol depth of weir.
- E = Length of heodwall extension.
- J = Height of wingwoii and sidewall at junction.



PERSPECTIVE VIEW

REFERENCE

Rev. 12-14-53

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

STANDARD DWG. NO.
ES-63
SHEET 1 OF 1
DATE 1-26-52

A3-Reference 4

Harding Lawson Associates

Engineering and Environmental Services

GEOTECHNICAL EVALUATION
W.R. GRACE DAM
RAINY CREEK, MONTANA

HLA Job No. 5891,053.03

by

A Report Prepared for

W. R. Grace & Company
Construction Products Division
P.O. Box 609
Libby, Montana 59923

GEOTECHNICAL EVALUATION
W.R. GRACE DAM
RAINY CREEK, MONTANA

HLA Job No. 5891,053.03

by

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February 3, 1992

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS iii

I INTRODUCTION 1

II PROJECT DESCRIPTION 3

III FIELD INVESTIGATION AND LABORATORY TESTING 5

IV DISCUSSION 7
- A. Material Characterization 7
  - 1. Tailings 7
  - 2. Embankment Soils 7
  - 3. Natural Foundation Soils 8
- B. Seismic Design Criteria 8
  - 1. Regional and Site Geology 8
  - 2. Seismicity 11
  - 3. Design Ground Motion 12
- C. Groundwater Considerations 13
- D. Static and Dynamic Design Considerations 14
  - 1. Consolidation Settlement 14
  - 2. Liquefaction Potential 14
  - 3. Stability of Slopes 15
  - 4. Deformation Analysis 18

V CONCLUSIONS AND RECOMMENDATIONS 19

VI REFERENCES 21

VII ILLUSTRATIONS 22

DISTRIBUTION

LIST OF ILLUSTRATIONS

Plate 1	Site Plan
Plates 2 through 11	Logs of Borings A-1 through A-10
Plate 12	Soil Classification Chart
Plate 13	Physical Properties Criteria for Soil Classifications
Plate 14	Physical Properties Criteria for Rock Classification
Plates 15 through 25	Particle Size Analysis Data
Plate 26	Plasticity Chart
Plates 27 through 30	Unconsolidated-Undrained Triaxial Compression Test Reports
Plate 31	Consolidation Test Data
Plate 32	Compaction Test Data
Plate 33	Geologic Map of Site and Vicinity
Plate 34	Stability Analysis for Water Approximately 500 feet from Embankment
Plate 35	Stability Analysis for Water at Face of Embankment
Plate 36	Schematic Section of Proposed Drainage/Monitoring Schemes

1 INTRODUCTION

This report presents the results of a geotechnical investigation and assessment performed by Harding Lawson Associates (HLA) of the long-term stability of the W. R. Grace Zonolite tailings retention dam in Libby, Montana. Our study was conducted for W. R. Grace Company, Construction Products Division, as part of the closure plans for the mining facility.

In the early 1970s, we performed a preliminary exploration and a detailed foundation investigation for the construction of the tailings retention dam. The results were presented in reports dated January 8, 1971 and August 18, 1971, respectively. We subsequently performed construction observation services for the project. In 1974, we conducted waste disposal studies for the mine tailings and presented the results in reports dated July 19, 1974 and September 30, 1974. Finally, we presented the results of a processing study in a report dated February 29, 1980.

The purpose of the current investigation was to determine whether the long-term stability of the dam conforms to the State of Montana requirements for closure. Primary concerns regarding performance of the dam included: 1) strength of the tailings and the potential for a significant downstream flow of tailings in a postulated dam failure, 2) potential for liquefaction of the tailings during the maximum credible earthquake (MCE)* and its potential effects on the stability of the dam, and 3) reliability and adequacy of the existing surface and internal drainage systems for a permanent embankment.

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* The maximum credible earthquake is the maximum event which, consistent with current knowledge, may ever be expected at the building site within the known geological framework.

The scope of work for this phase of the study, as outlined in our revised proposal dated April 10, 1991, was to review pertinent literature and reports, explore subsurface conditions of the tailings, embankment, and foundation materials, and perform engineering analyses to develop conclusions and, as appropriate, recommendations regarding the following:

1. Seismic design criteria
2. Geotechnical characteristics of the tailings material.
3. Liquefaction potential of the tailings and foundation soil based on current standards of practice.
4. Long-term static and dynamic stability of the dam.
5. Adequacy of the existing internal drainage system of the dam.

II PROJECT DESCRIPTION

W.R. Grace dam is located on Rainy Creek, approximately 3 miles northeast of the Kootenai River and northwest of Vermiculite Mountain on which the mine and mill were formerly situated. Construction of the tailings retention dam began with a 50-foot-high starter embankment, which was completed in November 1971. Since that time, the storage has been incrementally increased using a downstream, staged method of construction to raise the embankment as the tailings accumulated. The embankment is now 127 feet high with the crest at approximate Elevation +2927 feet.* The tailings surface elevation adjacent to the dam varies between +2913 and +2908 feet and slopes down to about Elevation +2903 feet where it intersects the surface of a pond upstream of the dam, as shown on the Site Plan, Plate 1. At the time of our field investigation (June 1991), the pond was about 750 feet from the dam. The maximum depth, surface elevation, and distance of the pond from the dam vary seasonally; however, it is important to note that presently water is not impounded directly behind the dam and that the excess water from the drainage basin and Rainy Creek is diverted around the reservoir through a 48-inch diameter corrugated metal pipe culvert and an intake structure near the tailings-water interface upstream of the dam.

To restore Rainy Creek to its natural state, two alternatives are being considered by the design team: 1) to maintain the water upstream of the dam at approximately Elevation +2510 feet by constructing a shallow diversion levee about 500 feet upstream of the dam and by replacing the existing culvert with a channel starting at the proposed levee location to discharge excess water through a new spillway to be constructed in the left abutment, and 2) to allow water to flood the existing tailings reservoir and be

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* National Geodetic Vertical Datum, NGVD.

impounded directly behind the dam. The impounded water would be discharged through a new spillway to be constructed in the left abutment.

III FIELD INVESTIGATION AND LABORATORY TESTING

We explored the subsurface conditions at the dam site by drilling 10 test borings at the locations shown on Plate 1. Borings A-1 through A-5 were drilled in the tailings to obtain soil samples for classification and laboratory testing. To augment available subsurface information and to obtain data needed for determining liquefaction potential and evaluating the natural foundation material (based on today's standards), Borings A-6 through A-10 were drilled on the downstream side, near the left abutment. The borings were drilled to depths between 21-1/2 and 77 feet using a truck-mounted rotary drilling rig. An open well piezometer was installed in Boring A-8. Boulders encountered during drilling of Borings A-6 through A-10 were cored using NX coring equipment. Our field engineer logged the borings and obtained samples for visual classification and laboratory testing. The soil types encountered were classified in accordance with ASTM D2487-85 based on visual-manual procedures as outlined in ASTM D2488-84. The boring logs are presented on Plates 2 through 11. The soil classification system that was used is presented on Plate 12. Physical properties criteria for soil and rock classifications are presented on Plates 13 and 14, respectively.

Soil samples were obtained using a Sprague and Henwood (S&H) split-barrel sampler (3.0-inch-outside diameter, 2.45-inch-inside diameter), a standard penetration test (SPT) sampler, and thin-walled Shelby tubes (3.0-inch outside diameter, 2.87-inch inside diameter). Both S&H and SPT samplers were driven by a 140-pound, automatic-trip hammer falling 30 inches. The number of blows required to drive the sampler the final 12 inches of an 18-inch drive were recorded. The S&H blows were converted to pseudo SPT N-values* to aid in comparison with published data. This conversion is only approximate and its reliability varies with soil type and sampling procedures. The pseudo SPT N-values obtained with the S&H sampler and N-values obtained with the SPT sampler are shown on the boring logs. Shelby tubes were used to obtain relatively undisturbed samples of silty material.

The soil samples were reexamined in our laboratory to confirm field classifications and to select representative samples for testing. Laboratory tests determined moisture content, dry density, Atterberg limits, gradation, percent passing the No. 200 sieve, unconsolidated-undrained triaxial shear strength, consolidation characteristics, and compaction characteristics. The laboratory test results are summarized on the boring logs in the manner described in the Key to Test Data shown on Plate 12.

Particle size (gradation) data are presented on Plates 15 through 25. Atterberg limits test data are presented on Plate 26. Shear strength test data are presented on Plates 27 through 30. Consolidation test data are presented on Plate 31, and compaction test data are presented on Plate 32.

* The SPT N-value is defined as the number of blows of a 140-pound hammer, falling freely through the height of 30 inches, required to drive a standard penetration test sampler (2-inch outside diameter, 1-3/8-inch shoe inside diameter, and 1-1/2-inch tube inside diameter) the last 12 inches of an 18-inch drive. For SPT procedures, see ASTM D1586-84.

IV DISCUSSION

A. Material Characterization

1. Tailings

Borings A-1 through A-5 were drilled in the tailings material. The tailings consist of interbedded layers of soft to stiff elastic silt (60%) and loose to medium dense poorly graded sands and silty sands (40%) with mica and pyrite flakes. Silt and sand layers generally slope down and away from the embankment, reflecting the fact that tailings were discharged at the five discharge locations shown on Plate 1.

On the basis of laboratory tests performed on elastic silt samples, we assigned a static undrained shear strength to this material which varies linearly from 50 pounds per square foot (psf) at the surface to 1900 psf at the foundation level (Elevation +2800 feet). Based on our experience with similar material (San Francisco Bay Mud and other plastic silts), we judge that during the design earthquake the shear strength of silts encountered will be temporarily reduced by about 30 percent due to pore water pressure build-up caused by earthquake-induced cyclic loading conditions. We assigned an average friction angle of 30 degrees to the sands and silty sands for static loading conditions. Based on an empirical method suggested by Seed and Harder (1990), we assigned a post-liquefaction residual undrained shear strength of 100 psf to this material (see Section IV.D.2 for discussion on liquefaction potential).

2. Embankment Soils

Embankment soils were encountered near the bottoms of borings A-1, A-2 and A-4. These soils consist of dense to very dense, well graded silty sands. On the basis of 1) data obtained during this investigatlon, 2) laboratory tests performed on representative samples during the 1971 study, and 3) correlation with published data, we assigned the embankment soils an effective friction angle of 37 degrees and cohesion values ranging between 50 and 500 psf. The upper bound effective cohesion value of 500 psf was obtained from unconsolidated undrained triaxial tests. Due to strain rate/creep effects, however, the field cohesion value could be somewhat lower. Therefore, consideration of a range in the cohesion value (as stated above) was deemed appropriate.

3. Natural Foundation Soils

Natural foundation soils encountered during both the 1971 and present explorations consist mainly of dense to very dense poorly graded gravels, dense to very dense poorly graded sands and moderately hard, friable pyroxenite bedrock, with abundant magnetite and pyrite.

B. Seismic Design Criteria

1. Regional And Site Geology*

The project site on Rainy Creek is in a region exposing pre-Cambrian age bedrock. The rocks, chiefly argillite and quartzite of the Belt Series, are folded in a series of broad open northwest-trending folds. In Rainy Creek, the rocks are intruded by basic plutonic igneous rocks consisting of pyroxenite and syenite. A geologic map of the site and vicinity is shown on Plate 33.

The terrain within the region is relatively flat, and is the result of long continued erosion until mid-Tertiary geologic time. During the Pliocene through mid-Pleistocene epochs, the area was subjected to faulting and uplift which resulted in renewed stream

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* Harding, Miller, Lawson & Associates, 1971. "Foundation Investigation and Engineering Analyses, Tailings Dam, W. R. Grace & Company, Construction Products Division, near Libby, Montana," dated August 18, 1971.

downcutting. This formed the basic existing drainage pattern. During the Pleistocene epoch, glacial action widened and deepened the valleys, including Rainy Creek valley. The valley of the Kootenai River was dammed by a glacier and a lake was formed which reached a surface elevation of about 2500 feet.

Intermittent active displacement of the faults developed during the Pliocene and early Pleistocene epochs. These displacements continue at a few locations in western Montana. However, there is no indication of active faulting at or near the site either in the geologic literature or from visible terrain features.

Bedrock underlying the dam and tailings pond areas consists of predominantly dark green to black pyroxenite which is generally fine grained and highly friable. The upper few feet of pyroxenite bedrock has physical characteristics not unlike those of a dense sand. In trench exposures on the canyon slopes, the pyroxenite contains thin, discontinuous, crushed zones. These crushed zones are planar and oriented approximately parallel to the canyon slope surface, suggesting that they are the result of glacial movement and/or gravity creep.

Syenite dikes, generally 6 inches or less in thickness, locally intrude into the pyroxenite. The syenite is generally quite hard and coarse-grained. Quartz-tremolite veins also cut the pyroxenite. The tremolite alteration is also accompanied by varying amounts of iron sulphides and oxides.

During the Pleistocene glaciation, the Rainy Creek valley was occupied by a glacier that produced a somewhat flattened valley bottom with rounded sides, in contrast to the typical V-shaped canyon that would result from stream erosion alone. As the glacier retreated upstream (possibly more than once), outwash alluvium was deposited in the valley bottom. The alluvium consisted of mixed silts, sands, and gravels, within which are zones with mnny large boulders of hard quartzite. Occasionally, the boulders reach a diameter of 4 to 5 feet. Where viewed in trench walls, the glacial outwash consists predominantly of fine- to coarse-grained gravels with about 10 percent or less of fine sands and silts. The gravels contain zones of very high porosity and permeability.

Zones of finer-grained alluvium consisting of clayey gravels to thinly laminated silts are found in the valley bottom near the right abutment. These sediments appear to be largely rock flour deposited in a lake formed during glaciation.

The right abutment slope is underlain by a thick blanket of glacial outwash and till, probably a lateral moraine, to approximately Elevation +2890 feet. The thickness of this zone varies from a few feet to about 40 feet. Near the top of the abutment slope, the alluvium consists of nearly horizontally bedded silty and sandy gravels overlain by approximately 6 feet of thinly laminated fine silt, possibly of lacustrine origin.

The left abutment slope, in contrast, is blanketed by a relatively thin (10 feet and less) mantle of slope debris and remnants of a lateral moraine near the base of the canyon slope. At about Elevation +2830 feet, there is a remnant of an outwash terrace capped by a horizontal, 4-foot-thick bed of highly permeable, relatively clean sand and gravel.

A 1- to 2-inch-thick bed of nearly white silt overlies the top of the glacial outwash gravels in the valley bottom, suggesting that there was a temporary lake in Rainy Creek at the close of the Pleistocene glaciation.

Recent unconsolidated alluvium in the valley bottom consists of a blanket of soft silt up to about 6 feet thick. This flood plain deposit locally contains fine sand and gravel streaks with occasional large boulders near the present stream course. These materials are reworked glacial outwash.

2. Seismicity

The project site is within a seismically active zone that forms an arc through western Montana, northwestern Wyoming, southeastern Idaho, and Utah. Earthquakes of both large and small intensities have an epicentral concentration within this seismic zone. In Montana, six earthquakes with intensities of VIII (Modified Mercalli Scale) or greater have occurred within this zone since 1868. Most of the strong historic earthquakes in Montana, including the Hebgen Lake earthquake which occurred in 1959, have occurred between Yellowstone National Park and Helena. The 1959 Hebgen Lake earthquake had no significant effect on the Libby area. There is no record of moderate to large earthquakes locally. However, smaller "random" events (events that are not related to known active or potentially active faults) with a maximum magnitude of 5 have occurred in the region.

There are several potentially active faults in the region that may affect the project site. These are the Lenia, Bull Lake, Savage Lake, O'Brien Creek, Snowshoe, Rock Lake, and Hope faults. The closest, the O'Brien Creek fault, is about 13 km west of the site (see Plate 33).

The Lenia fault has been traced continuously for 16 km within the Libby quadrangle. It almost certainly continues southward, either under the Bull Lake, or less probably, through the Trio prospect. Evidence suggests that movement along the fault is vertical.

The Bull Lake fault is a normal fault with younger beds exposed in the relatively downthrown block on the west. It has been traced for 21 km along a curving course, concave to the east. The fault is not observed north of Madge Creek, and is joined by the Savage Lake fault south of Crowell Creek. From this junction, the Bull Lake fault trends southward and then curves to the southeast.

The Savage Lake fault is largely inferred from physiographic and structural evidence. The fault, seen at the Carter prospect about 2 km east of Savage Lake, trends first, southwestward, then southward along the east side of the Libby Formation.

The O'Brien Creek fault is a normal fault thought to pass through the "island" of Wallace Formation just north of Savage Lake in a north-northwesterly fashion and cross beneath the Great Northern tracks. The fault is concealed under alluvium north of the Kootenai River. It then passes very close to the mountain front on the east side of O'Brien Creek.

The Snowshoe fault strikes north and is nearly vertical for most of its trace of 26 km. It cuts the crest of the northward-plunging Snowshoe anticline and is cut off at the north by a small fault along Horse Creek. The fault terminates at the Snowshoe anticline east of Elephant Peak.

The Rock Lake fault, approximately 19 km long, extends from Dad Peak southeastward past Wanless Lake. Displacement along this fault has been irregular in magnitude and direction, but movement has been essentially vertical. The Hope fault, a normal fault with the downthrow on the southwest, was traced from Hope, Idaho, to Heron, Montana.

On the basis of fault length and published relationships correlating fault rupture length and earthquake magnitude, we have conservatively assigned a maximum credible magnitude of 7 to these faults.

3. Design Ground Motion

Based on a postulated magnitude 7 earthquake on the nearby O'Brien Creek fault (about 13 km from the site) and using published attenuation relationships (Idriss, 1987), we estimate the peak bedrock acceleration (PBA) at the site to be about 0.30 gravity (g). The estimated PBA corresponding to a "random" magnitude 5 event at a close distance (less than 1 km) is less than 0.30g. The Uniform Building Code (UBC, 1991) maps the project site in the Seismic Zone 2B with the expected PBA of 0.20g. We believe that a 0.30g acceleration conservatively envelopes expected long-term seismic activities in the region.

C. Groundwater Considerations

Because we used a rotary wash drilling system, the groundwater level could not be measured during our field investigation. However, stabilized groundwater table was measured about 10 feet below ground surface from the piezometer installed in Boring A-8. In addition, groundwater level data are available from five piezometers installed following the construction of the starter dam in 1971 (see Plate 1 for piezometer locations). Since then, groundwater level has been monitored by W. R. Grace staff on a monthly basis. Water levels observed in piezometers indicate that water flows mainly in the highly previous natural gravelly foundation material and that the phreatic surface rises only occasionally above the foundation level (one of the six piezometers indicates temporary water levels that were about 3 feet above the dam foundation).

During the construction of the starter embankment, a series of 20-foot-wide, 2-foot-high drainage blankets consisting of native gravel material was placed at the embankment foundation level. Eight-inch-diameter perforated pipes were embedded within the drainage blankets to collect the water and transport it to the downstream side. These pipes were connected to a single 14-inch-diameter pipe which was extended at each subsequent stages of construction and presently emerges from the downstream face near Boring A-8. The volume of discharged water was roughly estimated to be 300 gallons per minute (gpm) at the time of this investigation.

D. Static and Dynamic Design Considerations

1. Consolidation Settlements

As described above, two scenarios are being considered for the restoration of Rainy Creek. If water is kept near the proposed levee, consolidation settlements will continue to occur in the tailings as water drains from the tailings material. The magnitude of settlement will vary with the thickness of the tailings. Where the tailings are 100 feet thick, we anticipate a total settlement of approximately 5 feet would occur over a 30-year period based on the consolidation characteristics of the tailings. We estimate that half of this settlement will occur during the next few years. On the other hand, if water is allowed to rise and pond against the embankment, the pore pressure would be unchanged, and therefore no additional settlement of the surface of tailings would occur.

2. Liquefaction Potential

Using field and laboratory test data to evaluate the potential for liquefaction of the tailings material, we determined that if the tailings remain saturated, the discontinuous loose sand and silty sand layers encountered in the tailings material will likely liquefy during the design earthquake (MCE). We further determined that settlements of up to several inches could result from liquefaction and/or densification of loose sands in the tailings. These settlements would likely be nonuniform because of the variable thicknesses and depths of the sand layers.

Due to their high plasticity indices, saturated silts encountered in the tailings are not susceptible to liquefaction. However, we believe that a build-up of pore water pressure (if the tailings reservoir remain saturated) during the design earthquake could result in a decrease of up to 30 percent in the static undrained shear strength of these soils. If the silts are drained, there would be no reduction in undrained shear strength due to cyclic loading conditions.

3. Stability of Slopes

Stability analyses of the embankment and tailings were performed for four different cases, using the Bishop's Modified Method of Slices. The method analyzes circular slip surfaces using conventional limit equilibrium theory to compute factors of safety for the slope geometry and soil parameters being considered.

The cases analyzed were:

Case I - Static analysis with the water about 500 feet upstream of the dam (Plate 34).

Case II - Dynamic (pseudo-static) analysis with the water about 500 feet upstream of the dam and with seismic coefficients k_s, of 0.10g, 0.15g and 0.20g. For this case a yield seismic coefficient k_y was also determined. (This coefficient yields a factor of safety, F.S., of 1.0) (Plate 34).

Case III - Static analysis with the water at the upstream face of the dam, at the tailings elevation (Plate 35).

Case IV - Dynamic (pseudo-static) analysis with the water at the upstream face of the dam, at the tailings elevation. Seismic coefficients of 0.10g, 0.15g and 0.20g were used and a yield coefficient k_y was also determined (Plate 35).

On the basis of engineering analysis and measured groundwater levels in the piezometers, we believe that the groundwater level immediately upstream of the embankment does not rise above the foundation level. Therefore, for our stability analyses, we have assumed that the water level is at the base of the embankment (Plates 34a and 34b). If water Is allowed to come in contact with the embankment, it could become partially saturated, as shown by the phreatic surface on Plates 35a and 35b, with water emerging from the downstream face of the embankment. As a worst-case condition, we assumed that the pervious natural gravel layer will be clogged at some time in the future and would no longer be effective in providing drainage as presently observed. In reality, the relatively high permeability of the foundation material likely will cause the phreatic surface to drop, and water may never emerge from the downstream face of the embankment.

Without a permanent monitoring system in place, however, it would be inadvisable to assume a long-term phreatic surface lower than that shown on Plate 35a and 35b. As indicated in our 1971 reports, the material used in construction of the dam is highly susceptible to erosion. Therefore, water emerging from the downstream face of the embankment and running parallel to surface would be unacceptable for both stability and erosion considerations. The problem could be mitigated either by 1) providing a permanent monitoring program, such as installing piezometers in the downstream face to monitor the actual location of the phreatic surface within the embankment, or 2) providing a new internal drainage system, such as a chimney drain constructed near the downstream toe to collect water which otherwise could cause erosion of the downstream face. Assuming one of these alternatives will be put in place to mitigate the potential for a "localized" instability and/or erosion, we evaluated the overall global static and dynamic stability of the two alternatives shown on Plates 34 and 35.

The analyses were performed considering a range of shear strength values appropriate for static and dynamic loading conditions. The shear strength values are summarized below:

i) Embankment Material

c' = 50 psf, ϕ'= 37 degrees (see Plates 34b and 35b)
c' = 500 psf, ϕ'= 37 degrees (see Plates 34a and 35a)

ii) Tailings

a) Elastic Silts

Su (static)	=	50	psf at 0 feet depth
		1900	psf at 100 feet depth

Su (dynamic) = 35 psf at 0 feet depth
1330 psf at 100 feet depth

b) Sands and Silty Sands

ϕ' (static) = 30 degrees
Su (dynamic) = 100 psf

iii) Foundation Material

ϕ' = 45 degrees

The results of the analyses are presented in the following table.

Case	Seismic Coefficient, k_s	Minimum Factor of Safety F.S	Yield Seismic Coefficient, k_y
I and II	0	1.83 - 2.28	--
	0.10	1.44 - 1.79	
	0.15	1.29 - 1.61	0.28 - 0.42
	0.20	1.17 - 1.46	
III and IV	0	1.74	--
	0.10	1.30	
	0.15	1.16	0.22
	0.20	1.05	

It should be noted that the slip surface shown on Plate 35b was not considered critical in our analyses because of the future installation of a new chimney or blanket drain. Either drain system would reduce the seepage and ravelling forces that would produce localized instability associated with the slip surface on Plate 35b.

The results of our stability analysis indicate that the dam is stable during both static and dynamic loading conditions. According to recommendations by Seed (1979), earth dams of similar construction are expected to experience limited deformations and remain stable during and after a magnitude 7 earthquake provided that 1) the build-up of pore water pressure does not significantly reduce the static strength of embankment material and 2) a minimum factor of safety of 1.15 is computed from a pseudo-static analysis using a seismic coefficient of 0.10g. The pseudo-static factors of safety computed for various phreatic surface locations and soil properties are well above the minimum required value. In addition, the embankment material is dense to very dense and is not susceptible to an appreciable loss of shear strength. Therefore, the dam is expected to remain stable during and following the design earthquake.

4. <u>Deformation Analysis</u>

To estimate the magnitude of permanent displacement of the embankment during the design earthquake, we performed studies using a simplified response and deformation analysis based on methods developed by Makdisi and Seed (1978, 1979).

First, the intensity of shaking within the embankment was estimated based on the estimated natural frequency of the embankment, anticipated frequency content of the input base motion, and dynamic soil properties of similar dam embankment material. The response spectra for rock sites by Idriss (1987) scaled to a PBA of 0.3g was used in our analysis. A maximum crest acceleration of about 0.8g was computed.

Next, based on the geometry and location of the critical slip surface, and a yield coefficient of 0.22 to 0.42g (see Plates 34 and 35), we calculated permanent deformations to be insignificant, which confirms conclusions by Seed (1979).

V CONCLUSIONS AND RECOMMENDATIONS

The results of field investigation and laboratory testing performed on the tailings material indicate that during a hypothetical embankment failure, the potential for a tailings material flow and contamination of the downstream area is very low because of the strength of the tailings as they now exist. We judge that the consistency of the tailings material is such that if a section of the embankment were removed, the tailings would fail but would maintain approximately a 4:1 (horizontal to vertical) slope.

On the basis of 1) the results of our 1971 studies, 2) geotechnical data obtained during our construction observation services, 3) piezometric data collected during about 20 years of operation, and 4) results of our present studies, we conclude that the embankment, in its present condition, is adequately safe during both static and maximum credible seismic loading conditions. We also judge that if the water were allowed to come in contact with the embankment, the dam would be safe under static and seismic loading conditions. However, due to potential problems discused below, we strongly recommend that the long-term performance of the dam be carefully monitored and mitigation measures be immediately implemented should the monitoring program indicate potential instability.

If, as part of restoration of the Rainy Creek, water is allowed to flood the tailings reservoir area and is maintained at the present tailings level, it could potentially emerge from the downstream face of the embankment causing erosion of soil and eventually, localized slope instability problems.

For the past 20 years, the drain pipe beneath embankment has successfully collected water at the dam foundation level and discharged it into the creek at the downstream side. This system is expected to operate effectively in the future. However, it is possible that the pipe may corrode or clog during long-term operation. If so, water could be released from the pipe within the embankment, and, if not absorbed by previous natural drainage material upon exit, erode the downstream face, eventually causing localized instability of the embankment. Although unlikely, the same problems could arise from clogging of the natural gravel drainage system.

An acceptable monitoring program would consist of the installation of new piezometers at locations shown on Plate 36 to detect the presence of water and the location of the phreatic surface inside the embankment. The piezometric data should be regularly reviewed and conditions of the dam be periodically inspected. Mitigating measures such as installation of a blanket drain, chimney drain, or other acceptable drainage system (see Plate 36) near the downstream toe should be adopted if and when water approaches the downstream face of the embankment.

If water is kept at Elevation +2910 feet about 500 feet upstream of the dam and a channel is constructed to collect Rainy Creek and flood water, nonuniform settlement of the tailings should be anticipated. Since a total long-term settlement of about 5 feet is anticipated as water drains out of the tailings material, the channel and its lining system should be flexible enough to tolerate the potentially large differential settlement. To minimize anticipated differential settlement, we recommend that the channel be constructed as close as possible to the left abutment at a starting elevation of about +2910 feet.

VI REFERENCES

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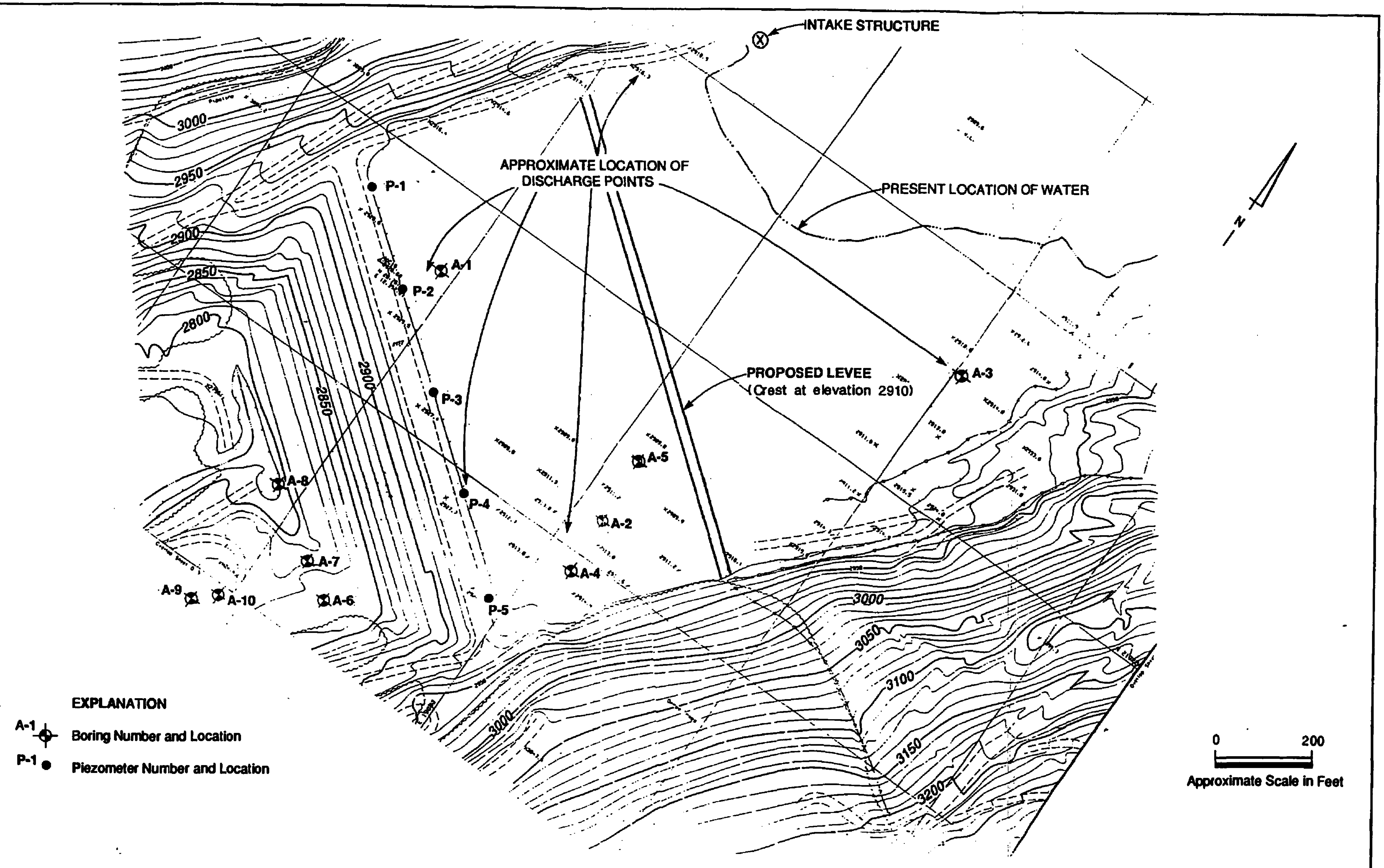
VII ILLUSTRATIONS



Reference: Topographic Map prepared by Walker and Associates, Inc. untitled, undated.

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Site Plan
W.R. Grace Dam
Rainy Creek, Montana

PLATE 1

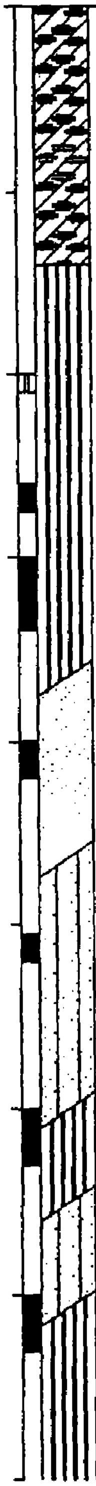
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	

135398

Equipment 4" dia. Rotary

Elevation 2912.9 ft** Date 06/27/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Description
				0	GRAY-BROWN SILTY GRAVEL WITH SAND (GM) moist
				5	ACCESS FILL
					OLIVE GREEN ELASTIC SILT (MH) soft to medium stiff, saturated, abundant fine grained platey minerals (diesel odor)
			14*	10	
			4*		
-200=7.7% MA, Sec Plate 15 TxUU 312(450) LL=60, PI=8 -200=75.9%	71.1	56	Push	15	color changes to blue-green with interbedded sandy silt and silty sand stringers (diesel odor)
					GREEN POORLY GRADED SAND (SP) loose, saturated, fine grained (diesel odor)
			4	20	
					OLIVE GREEN SILTY SAND (SM) loose, saturated (diesel odor)
-200=45.7 MA, See Plate 16			4*	25	
			6	30	OLIVE GREEN ELASTIC SILT (MH) medium stiff, saturated, abundant fine pyrite flakes (diesel odor)
					OLIVE-GREEN SILTY SAND (SM) loose, saturated, fine-grained, abundant mica
	59.7	67	7*	35	OLIVE GREEN ELASTIC SILT (MH) medium stiff, saturated, moderately plastic
				40	



*Blow counts converted to pseudo-standard pentration blow counts using a conversion factor of 0.6.

**Elevation referenced to National Geodetic Vertical Datum.

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Log of Boring A- 1 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **2a**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample	Equipment 4" dia. Rotary Elevation 2912.9 ft** Date 06/27/91
			8	40		OLIVE GREEN POORLY GRADED SAND (SP) loose, saturated, fine-grained, angular to platey, trace silt and gravel (diesel odor)
			13*	45		becoming medium dense
			38			MOTTLED GRAY-GREEN WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM) very dense, saturated, fine to coarse grained sand angular to subrounded gravel, with abundant platey minerals
			30/5.6**	50		(Embankment Material)
-200=8.4% MA, See Plate 17	12.9	132	30/5**	55		
						Boring was terminated at 56.0 feet. No free groundwater was encountered.
				60		
				65		
				70		
				75		
				80		

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Log of Boring A- 1 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **2b**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2911.4 ft** Date 06/26/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Description
				0	GRAY-BROWN SILTY GRAVEL WITH SAND (GM) moist (auger cuttings) — ACCESS FILL
				5	DARK GRAY-GREEN SILTY SAND (SM) very loose, saturated, fine-grained, platey grains, with abundant pyrite
			2*	10	
					GRAY-GREEN ELASTIC SILT (MH) soft, saturated, trace fine-grained sand, moderate to high plasticity
			3	15	DARK GREEN SILTY SAND (SM) very loose, saturated, fine-grained, platey grains
					OLIVE GREEN ELASTIC SILT (MH) soft, saturated, trace fine pyrite flakes, with occasional thin stringers of silty sand
			1*	20	
TxUU 3S3(500), See Plate 28 LL=51, PI=10, See PLate 26 -200=71.3%	63.2	62	Push	25	
			6	30	becoming medium stiff, with interbedded brown silty sand stringers
-200=75.3%	67.8	64	5*	35	BROWN AND OLIVE GREEN SILTY SAND (SM) loose, saturated, fine platey grained
					OLIVE GREEN ELASTIC SILT (MH) stiff, saturated, high plasticity, trace platey pyrite
				40	

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Log of Boring A- 2 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **3a**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2911.4 ft** Date 06/26/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Description
			10	40	
TxUU 946(800), See Plate 29 LL=71, PI=27, See Plate 26 -200=84.5% CC=0.75	60.4	64	Push	45	OLIVE GREEN SANDY ELASTIC SILT (MH) stiff, saturated, with abundant fine platey minerals, fine-grained sand with interbedded silly sand seams
-200=33.5% MA, See Plate 18	47.4	77	12*	50	DARK GREEN SILTY SAND (SM) medium dense, saturated abundant platey minerals
			14	55	DARK GREEN POORLY GRADED SAND WITH SILT (SP-SM) medium dense, saturated, fine to medium grained, angular to platey grains, with occasional sandy silt stringers
				60	OLIVE GREEN ELASTIC SILT (MH) stiff, saturated, abundant platey minerals, moderately plastic
			13		
			23	65	GREEN SILTY SAND (SM) medium dense, saturated, pockets of sandy silt
				70	
			21	75	DARK GREEN POORLY GRADED SAND WITH SILT (SP-SM) medium dense, saturated, fine grained sand, platey grains (Embankment Mateiial)
				80	Boring was terminated at 77.0 feet. No free groundwater was encountered.

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Log of Boring A- 2 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **3b**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2913.5 ft** Date 06/28/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Description
				0	GREEN WELL GRADED SAND WITH GRAVEL (SW) moist (auger cuttings) — ACCESS FILL
					GREEN WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM) loose, saturated, coarse grained sand, angular to platey grains, angular gravel
-200=9.7% MA, See Plate 19			3	5	
					OLIVE GREEN ELASTIC SILT WITH SAND (MH) very soft, saturated, moderate to high plasticity
				10	
			1*		
			1		
					SILTY SAND (SM) loose, saturated, fine to medium grained
					OLIVE GREEN ELASTIC SILT (MH) very soft, saturated, trace fine grained sand
				15	
				20	
			2*		OLIVE GREEN SILTY SAND (SM) very loose, saturated, fine grained, platey
			1		
				25	
-200=76%				30	BLUE-GREEN ELASTIC SILT (MH) soft to stiff, saturated, trace pyrite, occasional thin stringers of sandy silt
			4*		
			S		
TxUU 1006(750), Sec Plate S0 LL=69, PI=2S, Sec Plate 26 -200=82.6%	64.0	62	Push	35	
			9		
				40	

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Log of Boring A- 3 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **4a**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2913.5 ft** Date 06/28/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample	Description
				40		
-200=6.0% MA, See Plate 20	14.0	128	18*	45		GREEN WELL GRADED SAND WITH GRAVEL (SW) dense, saturated, fine to coarse grained sand, gravel to 1 inch diameter
						Boring was terminated at 46.5 feet. No free groundwater was encountered.
				50		
				55		
				60		
				65		
				70		
				75		
				80		

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Log of Boring A- 3 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

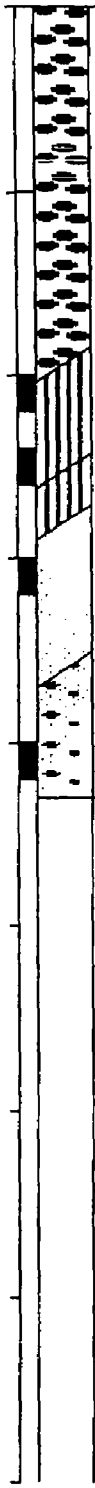
PLATE
4b

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G!9		

Equipment 4" dia. Rotary

Elevation 2913.6 ft** Date 06/27/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)
				0
			9*	5
-200=1.7 MA, See Plate 21 -200=6.1 MA, See Plate 22	31.7	84	5*	10
			1	
			7*	15
			16	20
				25
				30
				35
				40



BROWN GRAY POORLY GRADED GRAVEL WITH SAND (CP)
moist

ACCESS FILL

OLIVE BLUE-GREEN ELASTIC SILT (MH)
soft to medium stiff, saturated, interbedded with silty sand stringers

OLIVE GREEN ELASTIC SILT (MH)
soft, saturated, abundant fine platey minerals, with interbedded sand stringers (diesel odor)

GREEN POORLY GRADED SAND WITH GRAVEL (SP) loose, saturated, fine to coarse grained subangular to platey sand, trace subrounded gravel (diesel odor)

GREEN WELL GRADED SAND (SW)
medium dense, moist, fine to coarse grained sand,
syenite and pyroxinite up to 1" diameter

(Embankment Material)

Boring terminated at 21.5 feet
No free groundwater was encountered.

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Log of Boring A- 4 (Sheet 1 of 1)
W. R. Grace Dam
Rainy Creek, Montana

PLATE
5

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2910.1 ft** Date 06/28/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample
				0	
				5	
			5*		
				10	
			2*		
				15	
			2*		
-200=5E.3 MA, See Plate 23			1*	20	
				25	
			4*		
			4*	30	
-200=76.6%	67.5	61	4*	35	
				40	

BROWN SILTY GRAVEL WITH SAND (GM)
moist

ACCESS FILL

DARK GREEN POORLY GRADED SAND WITH SILT (SP-SM)
loose, saturated, fine to medium grained, platey minerals (diesel odor)

OLIVE BLUE-GREEN ELASTIC SILT (MH)
very soft, saturated, abundant pyrite minerals, with pockets of silty sand

BROWN AND OLIVE-GREEN SILTY SAND (SM)
very loose, saturated, occasional stringers of sandy silt

GREEN SANDY ELASTIC SILT (MH)
medium stiff, saturated, fine grained sand, stringers of silty sand

BROWN-GREEN POORLY GRADED SAND (SP)
very loose, saturated, fine grained

BLUE-GREEN ELASTIC SILT (MH)
medium stiff, saturated, trace pyrite flakes

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Log of Boring A- 5 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE 6a

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Blows/foot | Depth (ft) | Sample

Equipment 4" dia. Rotary

Elevation 2910.1 ft** Date 06/28/91

Laboratory Tests	Blows/foot	Depth (ft)	Description
		40	
	5		**BROWN AND GREEN SILTY SAND (SM)** loose, saturated, with interbedded layers of medium stiff elastic silt
-200=60.2% MA, See Plate 24	Push	45	
		50	Boring was terminated at 50.0 feet. No groundwater was encountered.
		55	
		60	
		65	
		70	
		75	
		80	

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Log of Boring A- 5 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **6b**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample	Equipment 4" dia. Rotary Elevation 2836.5 ft** Date 06/24/91
				0		GRAY AND BROWN POORLY GRADED GRAVEL WITH SAND (GP) dense, moist, cobbles up to 2' dia. with trace silt, subrounded to subangular
				5		
				10		
				15		
				20		occasional seams of syenite, tremolite, and quartz in boulders
				25		
				30		
				35		
				40		

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Log of Boring A- 6 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

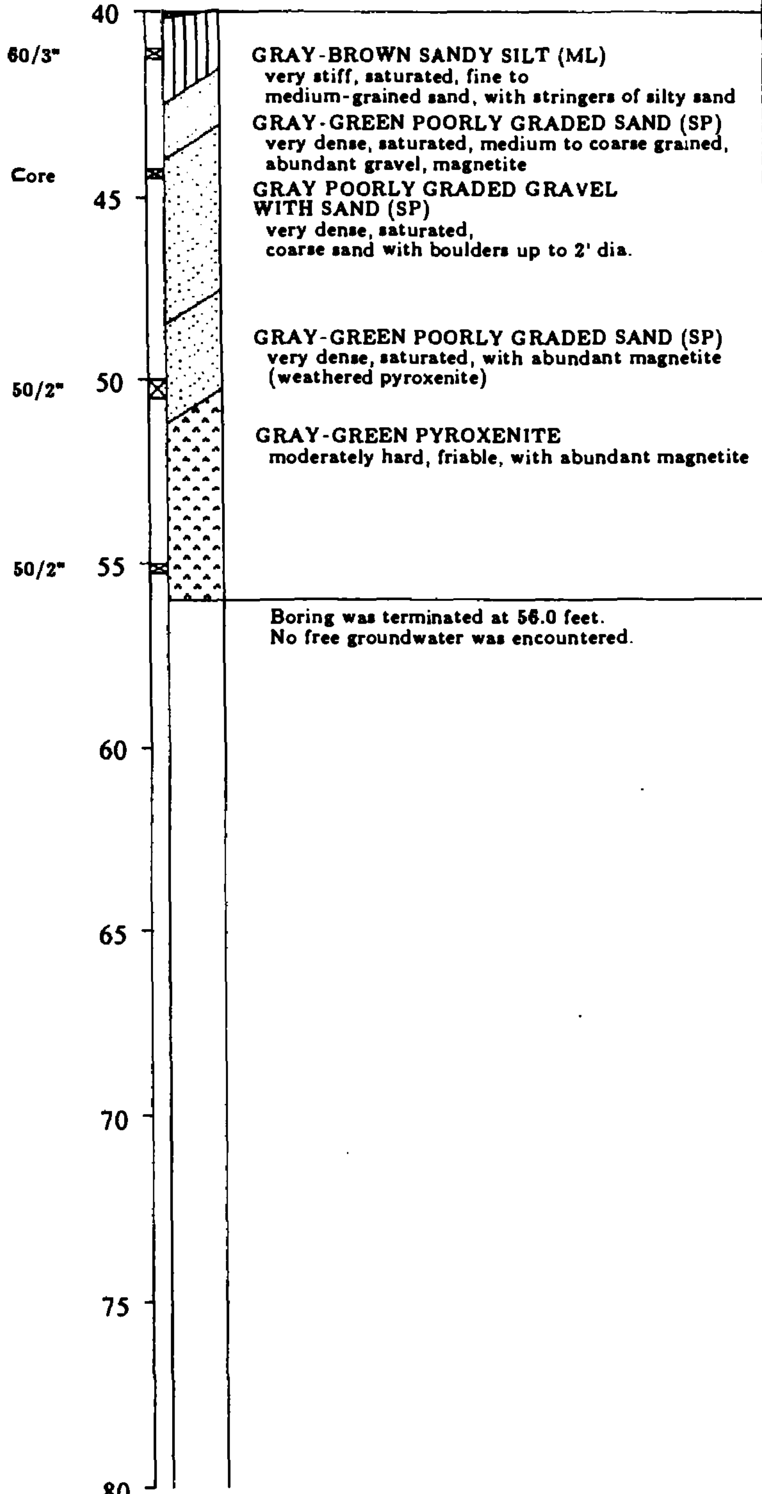
PLATE **7a**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2836.5 ft** Date 06/24/91

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Blows/foot | Depth (ft) | Sample



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Log of Boring A- 6 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **7b**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2816.0 ft** Date 06/25/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample	Description
				0		GRAY POORLY GRADED GRAVEL WITH SAND (GP) moist — ACCESS FILL
						GRAY-BROWN POORLY GRADED GRAVEL WITH SAND (GP) dense, saturated, coarse grained sand, subrounded to angular gravel
				5		
				10		
				15		
			43		X	
				20		
				25		
			50/5"		X	GRAY-GREEN POORLY GRADED SAND (SP) very dense, saturated, medium grained, with abundant magnetite crystals and pyrite flakes with seams of syenite and tremolite
				30		
			50/1"	35	X	GREEN PYROXENITE moderately hard, friable, with abundant magnetite and pyrite
						Boring was terminated at 35.0 feet. No free groundwater was encountered.
				40		

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Log of Boring A- 7 (Sheet 1 of 1)
W. R. Grace Dam
Rainy Creek, Montana

PLATE
8

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2791.3 ft** Date 06/29/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample	Description
				0		GRAY POORLY GRADED SAND WITH GRAVEL (SP) moist (auger cuttings) — ACCESS FILL
						GRAY-BROWN POORLY GRADED GRAVEL WITH SILT (GP) dense, saturated, cobbles up to 1' dia. (auger cuttings)
				5		
				10		
				15		
			50/5"	20		becoming very dense
			50/4"			GRAY-GREEN POORLY GRADED SAND WITH GRAVEL (SP) very dense, saturated, fine to medium grained sand
			50/1"	25		
			50/5.5"			GRAY POORLY GRADED GRAVEL WITH SILT (GP) very dense, saturated
				30		Boring was terminated at 30.0 feet. No free groundwater was encountered.
				35		
				40		

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Log of Boring A- 8 (Sheet 1 of 1)
W. R. Grace Dam
Rainy Creek, Montana

PLATE 9

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Equipment 4" dia. Rotary

Elevation 2828.1 ft** Date 06/30/91

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Description
				0	**BROWN SILTY SAND WITH GRAVEL (SM)** loose, moist, with cobbles to 6" dia.
-200=28.3% MA, See Plate 25	12.5	133	9	10	
					becoming very loose
			3		
			18		becoming medium dense
			25	15	wood pieces up to 1" at 15.2 feet
			10		
					GRAY POORLY GRADED GRAVEL WITH SAND (GP)
			50/1"	20	dense, saturated, fine to medium grained sand
				25	

Boring was terminated at 25.5 feet after hitting obstruction.
No free groundwater was encountered.

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

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Log of Boring A- 9 (Sheet 1 of 1)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **10**

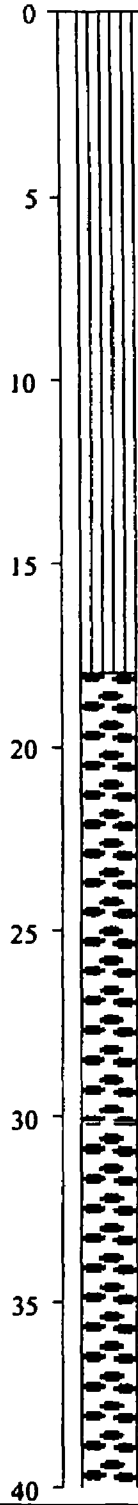
DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Laboratory Tests

Moisture Content (%)	Dry Density (pcf)	Blows/foot	Depth (ft)	Sample

Equipment 4" dia. Rotary

Elevation 2828.5 ft** Date 06/30/91



BROWN SANDY SILT WITH GRAVEL (ML)
soft to stiff, saturated, cobbles up to 6", occasional thin stringers of silty sand up to 1/2" thick (auger cuttings)

BROWN-GRAY POORLY GRADED GRAVEL WITH SILT (GP)
dense, saturated

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Log of Boring A-10 (Sheet 1 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE **11a**

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Blows/foot | Depth (ft) | Sample

Equipment 4" dia. Rotary

Elevation 2828.5 ft** Date 06/30/91

40

45

GRAY-BROWN WELL GRADED GRAVEL WITH SAND AND SILT (GW-GM)

50

55

GREEN PYROXENITE
moderately hard, friable

Boring was terminated at 57.0 feet.
No free groundwater was encountered.

60

65

70

75

80

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Log of Boring A-10 (Sheet 2 of 2)
W. R. Grace Dam
Rainy Creek, Montana

PLATE
11b

DRAWN	JOB NUMBER	APPROVED	FILE	DATE	REVISED DATE
	5891,053.03		11529G19		

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487-85

MAJOR DIVISIONS				GROUP NAMES
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	Clean gravels less than 5% fines	GW	WELL-GRADED GRAVEL, WELL-GRADED GRAVEL WITH SAND
			GP	POORLY-GRADED GRAVEL, POORLY-GRADED GRAVEL WITH SAND
		Gravels with more than 12% fines	GM	SILTY GRAVEL, SILTY GRAVEL WITH SAND
			GC	CLAYEY GRAVEL, CLAYEY GRAVEL WITH SAND
	SANDS 50% or more of coarse fraction passes No. 4 sieve	Clean sand less than 5% fines	SW	WELL-GRADED SAND, WELL-GRADED SAND WITH GRAVEL
			SP	POORLY-GRADED SAND, POORLY-GRADED SAND WITH GRAVEL
		Sands with more than 12% fines	SM	SILTY SAND, SILTV SAND WITH GRAVEL
			SC	CLAYEY SAND, CLAYEY SAND WITH GRAVEL
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50%		ML	SILT, SILT WITH SAND OR GRAVEL, SANDY OR GRAVELLY SILT
			CL	LEAN CLAY, LEAN CLAY WITH SAND OR GRAVEL, SANDY OR GRAVELLY LEAN CLAY
			OL	ORGANIC SILT OR CLAY, ORGANIC SILT OR CLAY WITH SAND OR GRAVEL, SANDY OR GRAVELLY ORGANIC SILT OR CLAY
	SILTS AND CLAYS Liquid limit 50% or more		MH	ELASTIC SILT, ELASTIC SILT WITH SAND OR GRAVEL, SANDY OR GRAVELLY ELASTIC SILT
			CH	FAT CLAY, FAT CLAY WITH SAND OR GRAVEL, SANDY OR GRAVELLY FAT CLAY
			OH	ORGANIC SILT OR CLAY, ORGANIC SILT OR CLAY WITH SAND OR GRAVEL, SANDY OR GRAVELLY ORGANIC SILT OR CLAY
HIGHLY ORGANIC SOILS			Pt	PEAT

For definition of dual and borderline symbols, see ASTM D2487-8S.

KEY TO TEST DATA

Perm	-	Permeability
Consol	-	Consolidation
LL	-	Liquid Limit (%)
Pi	-	Plasticity Index (%)
Gs	-	Specific Gravity
MA	-	Particle Sise Analysis
■	-	"Undisturbed" Sample
⊠	-	Bulk or Classification Sample
▥	-	Lost Sample

Shear Strength (psf) — Confining Pressure

TxUU	3200	(2600)	- Unconsolidated-Undrained Triaxial Shear
	(FM) or (S)		(field moisture or saturated)
TxCU	3200	(2600)	- Consolidated-Undrained Triaxial Shear
	(P)		(with or without pore pressure measurement)
TxCD	3200	(2600)	- Consolidated Drained Triaxial Shear
SSCU	3200	(2600)	- Simple Shear Consolidated Undrained
	(P)		(with or without pore pressure measurement)
SSCD	3200	(2600)	- Simple Shear Consolidated Drained
DSCD	2700	(2000)	- Consolidated Drained Direct Shenr
UC	470		- Unconfined Compression
LVS	700		- Laboratory Vane Shear
TV	800		- Torvane Shear
PP	400		- Pocket Penetrometer (actual reading divided by 2)

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Soil Classification Chart and Key to Test Data
W.R. Grace Dam
Rainy Creek, Montana

PLATE **12**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03		12/91	

138380

Relative Density of Coarse-Grained Soils

Relative Density	Standard Penetration Test Blow Count (blows per foot)
very loose	<4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	>50

Consistency of Fine-Grained Soils

Consistency	Identification Procedure	Approximate Shear Strength (psf)
Very soft	Easily penetrated several inches with fist	less than 250
Soft	Easily penetrated several inches with thumb	250-500
Medium stiff	Penetrated several inches by thumb with moderate effort	500 - 1000
Stiff	Readily indented by thumb, but penetrated only with great effort	1000 - 2000
Very Stiff	Readily indented by thumb nail	2000 - 4000
Hard	Indented with difficulty by thumb nail	greater than 4000

Natural Moisture Content

Dry - Requires considerable moisture to obtain optimum moisture content* for compaction

Moist - Near the optimum moisture content for compaction

Wet - Requires drying to obtain optimum moisture content for compaction

Saturated - Near or below the water table, from capillarity, or from perched or ponded water

* Optimum moisture content as determined in accordance with ASTM Test Method D1557-78.

Where laboratory data are not available, the above field classifications provide a general indication of material properties; the classifications may require modification if laboratory tests are subsequently conducted.

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Physical Properties Criteria for Soil Classifications
W.R. Grace Dam
Rainy Creek, Montana

PLATE **13**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	

I CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

II BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness (feet)	Stratification
Massive	Greater than 4.0	Very thick bedded
Blocky	2.0 to 4.0	Thick bedded
Slabby	0.3 to 2.0	Thin bedded
Flaggy	0.05 to 0.2	Very thin bedded
Shaly or platy	0.01 to 0.05	Laminated
Papery	Less than 0.01	Thinly laminated

III FRACTURING

Intensity	Size of Pieces (Feet)
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

IV HARDNESS

1. Soft - Reserved for plastic material alone.
2. Low hardness - Can be gouged deeply or carved easily with a knife blade.
3. Moderately hard - Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. Hard - Can be scratched with difficulty; scratch produces little powder and is often faintly visible.
5. Very hard - Cannot be scratched with knife blade; leaves a metallic streak.

V STRENGTH

1. Plastic or very low strength.
2. Friable - Crumbles easily by rubbing with fingers.
3. Weak - An unfractured specimen of such material will crumble under light hammer blows.
4. Moderately strong - Specimen will withstand a few heavy hammer blows before breaking.
5. Strong - Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. Very strong - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

VI WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates, and/or clay or silt.
M. Moderate - Slight changes or partial decomposition of minerals; little disintegration; cementation little to unaffected; moderate to occasionally intense discoloration; moderately coated fractures.
L. Little - No megascopic decomposition of minerals; little or no effect on normal cementation; slight and intermittent, or localized discoloration; few stains on fracture surfaces.
F. Fresh - Unaffected by weathering agents; no disintegration or discoloration; fractures usually less numerous than joints.

ANDESITE, BASALT, RHYOLITE
GRAYWACKE
SCHIST
SILTSTONE, MUDSTONE CLAYSTONE
CHERT
LIMESTONE, CORAL
SERPENTINE
TUFF
CONGLOMERATE
SANDSTONE
SHALE
PYROXENITE

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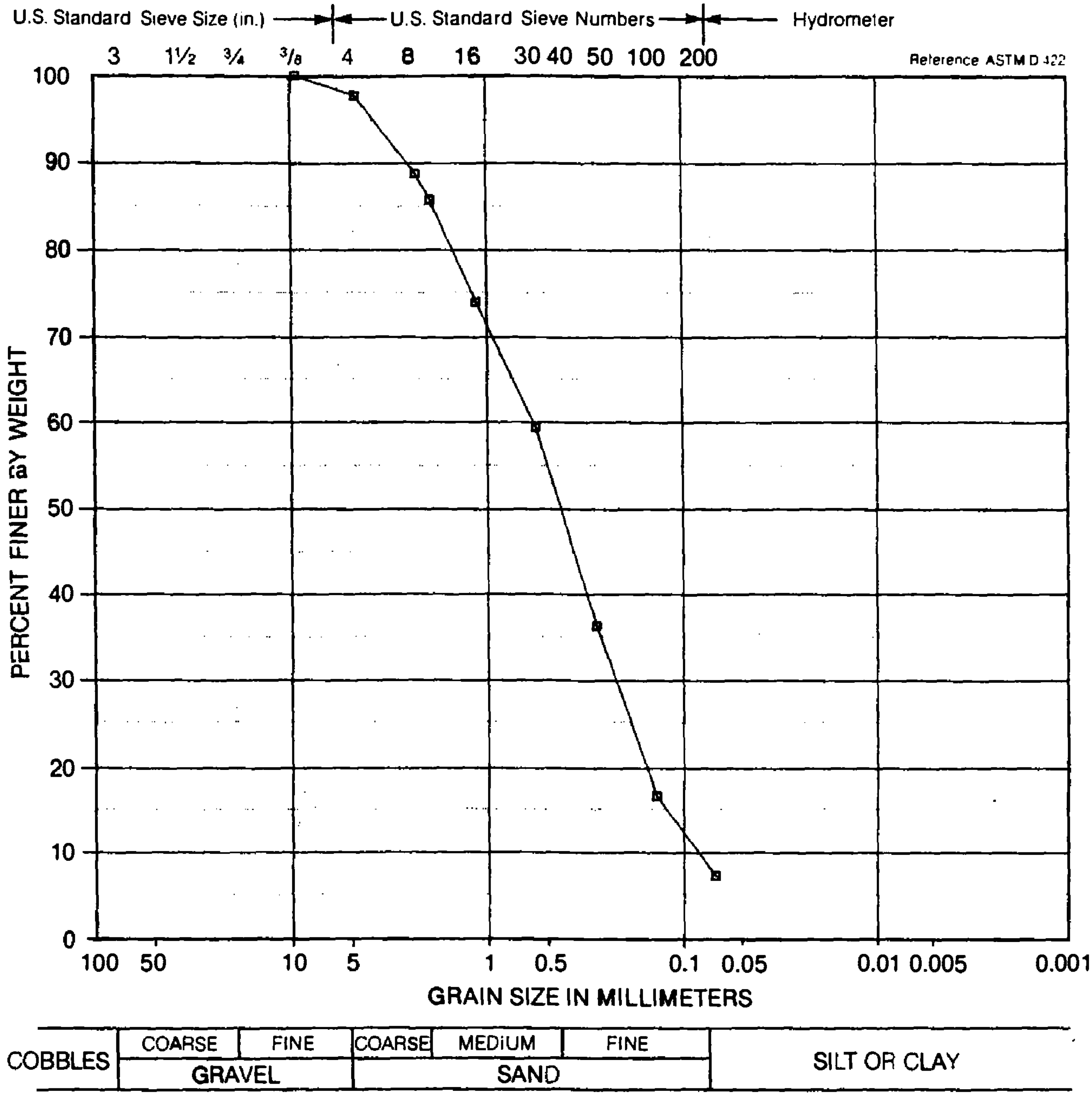
Physical Properties Criteria for Rock Classifications
W.R. Grace Dam
Rainy Creek, Montana

PLATE
14

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	



Symbol	Sample Source	Classification
▫	A-1 @ 15.0 FT	GRAY SAND W/ SILT (SW-SM)

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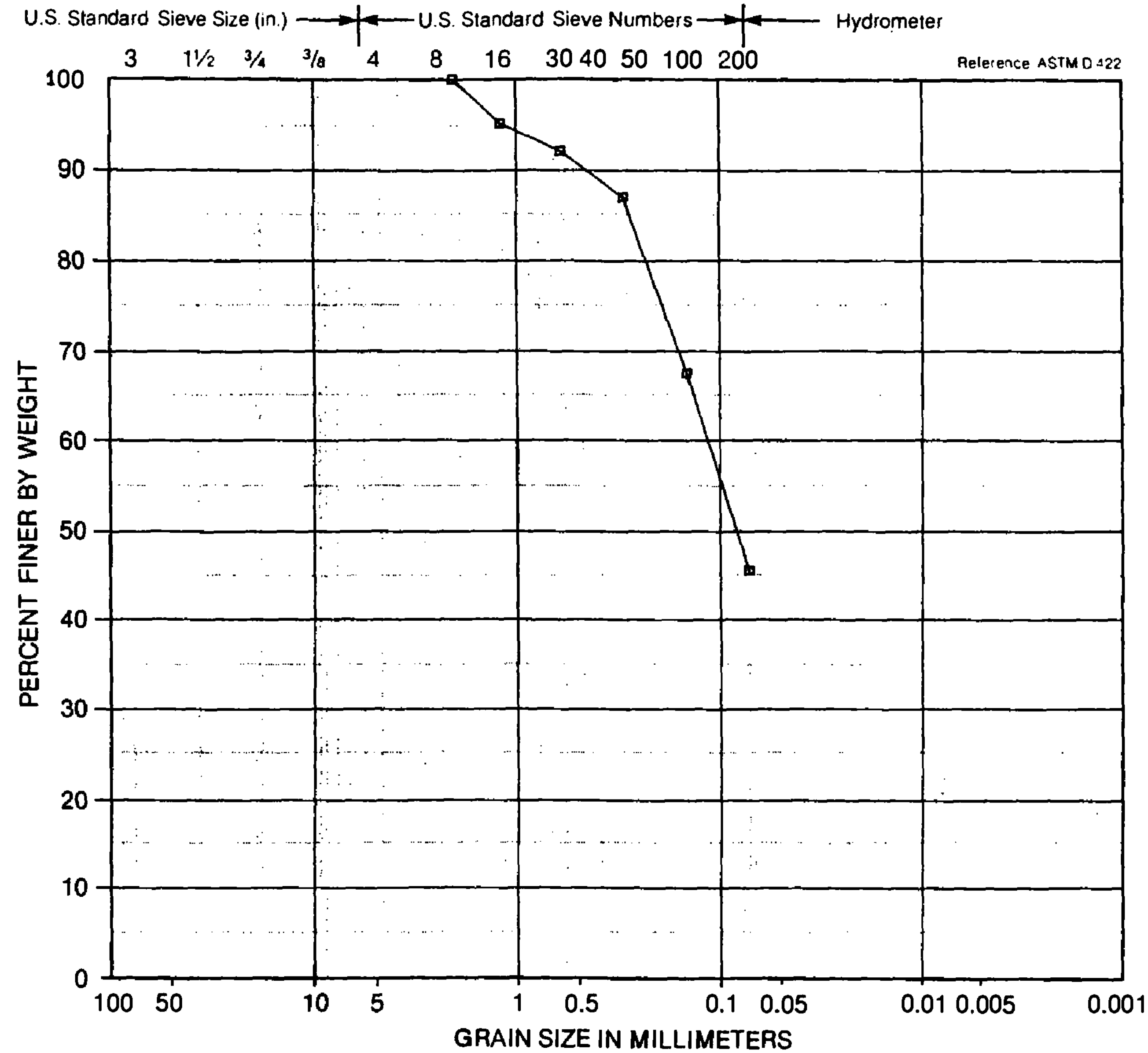
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
15

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	GH	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▫	A-1 @ 25.0 FT	OLIVE-GREEN SILTY SAND (SM)

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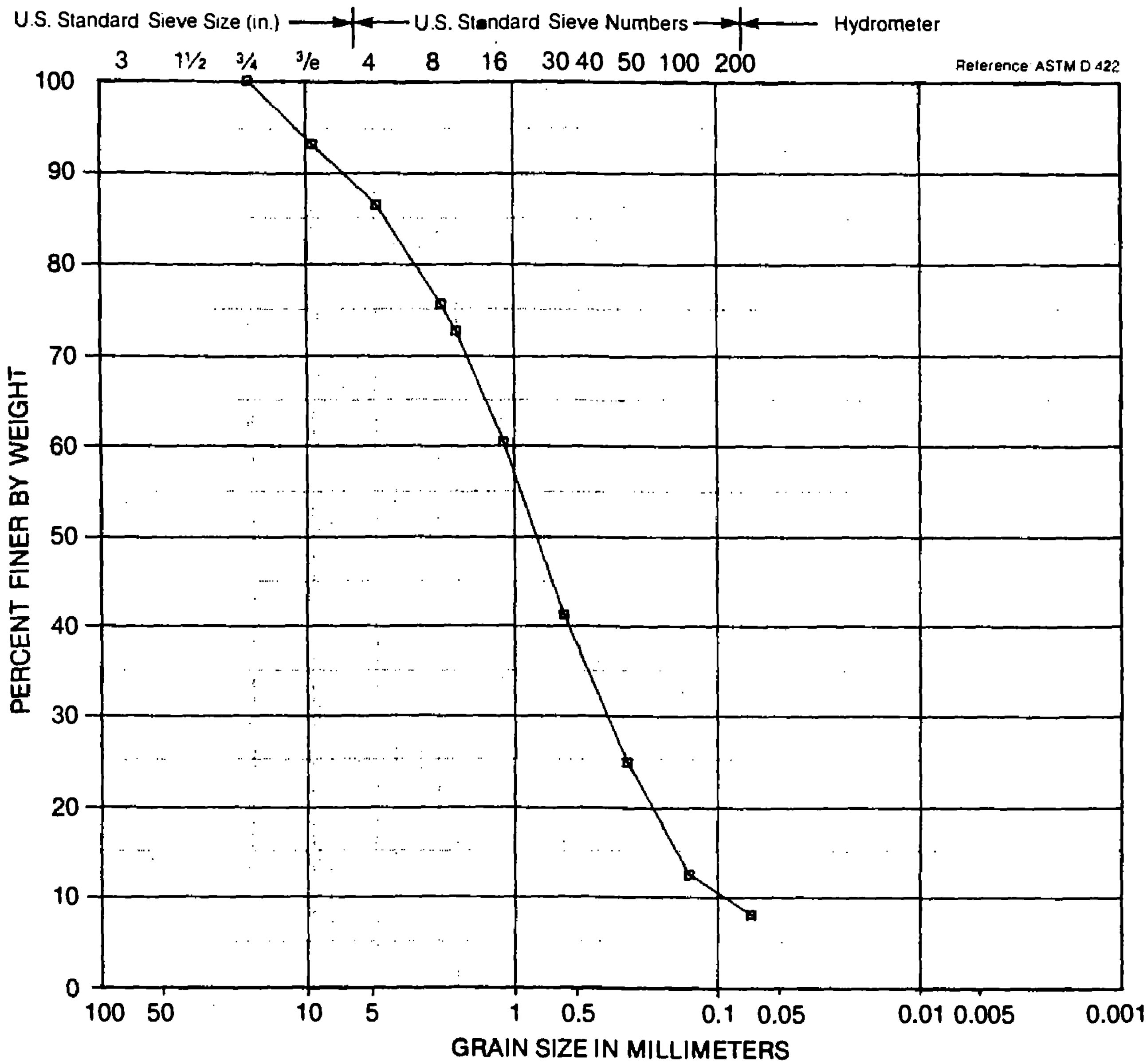
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
16

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	BH	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
⊠	A-1 @ 55.0 FT	MOTTLED GRAY-GREEN SAND W/ SILT (SW-SM)

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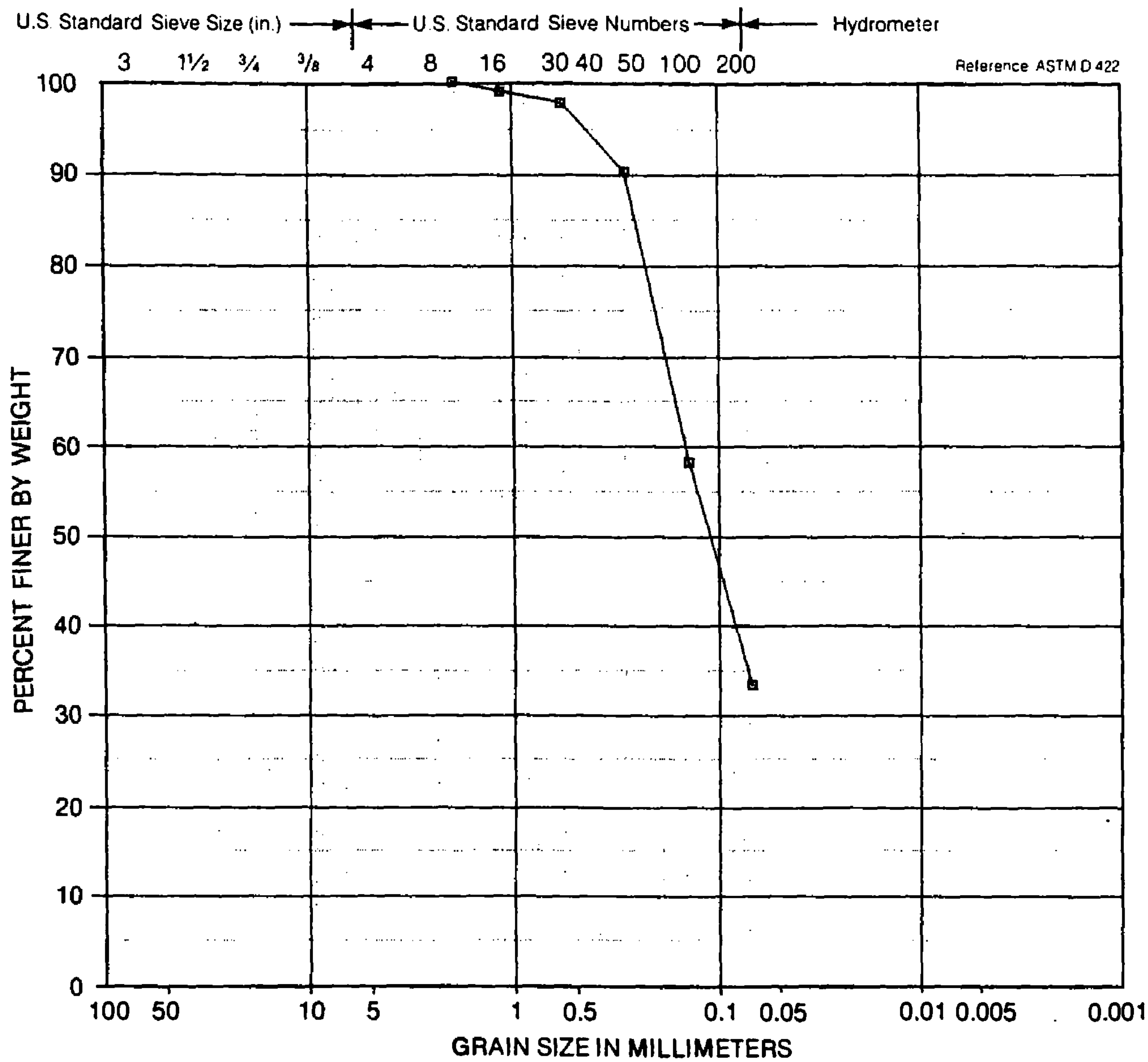
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
17

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	[illegible]	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▪	A-2 @ 50.5 FT	DARK GREEN SILTY SAND (SM)

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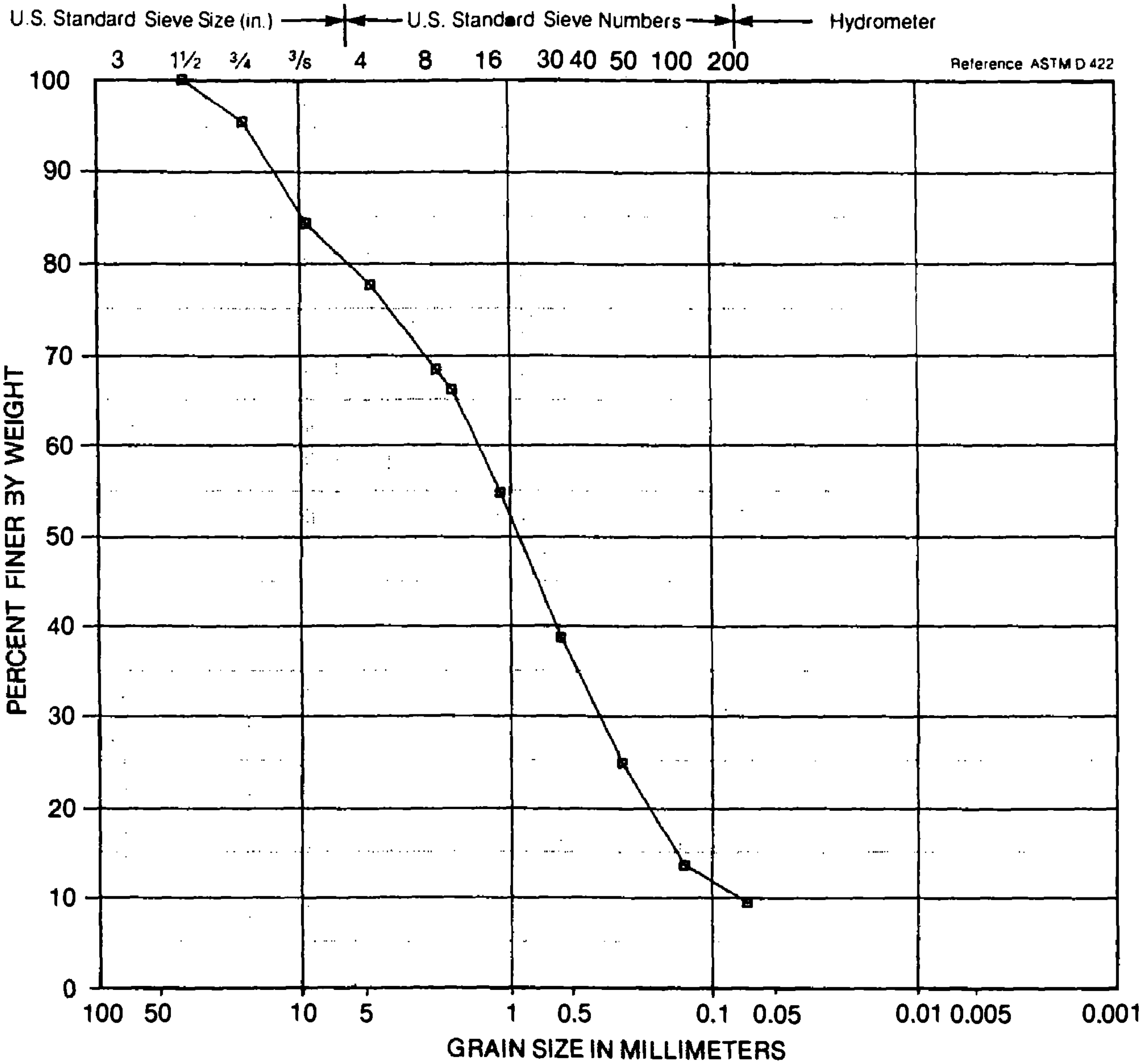
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
18

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03		07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▣	A-3 @ 5.0 FT	GREEN SAND W/ SILT AND GRAVEL (SW-SM)

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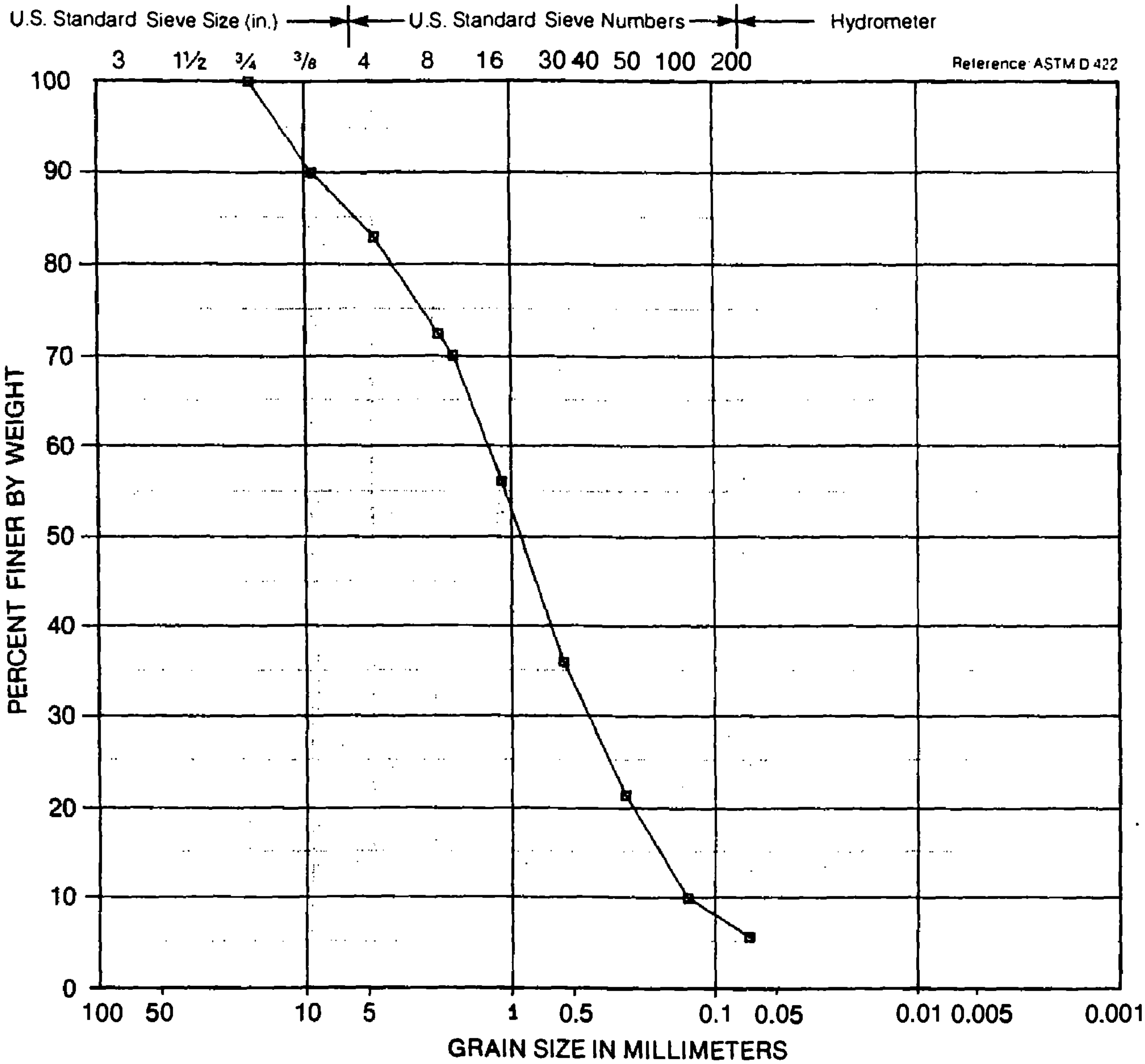
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
19

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5691.053.03		07-15-1991		11/91



COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT OR CLAY
	GRAVEL		SAND			

Symbol	Sample Source	Classification
▫	A-3 @ 45.5 FT	GRAY SAND W/ GRAVEL (SW)

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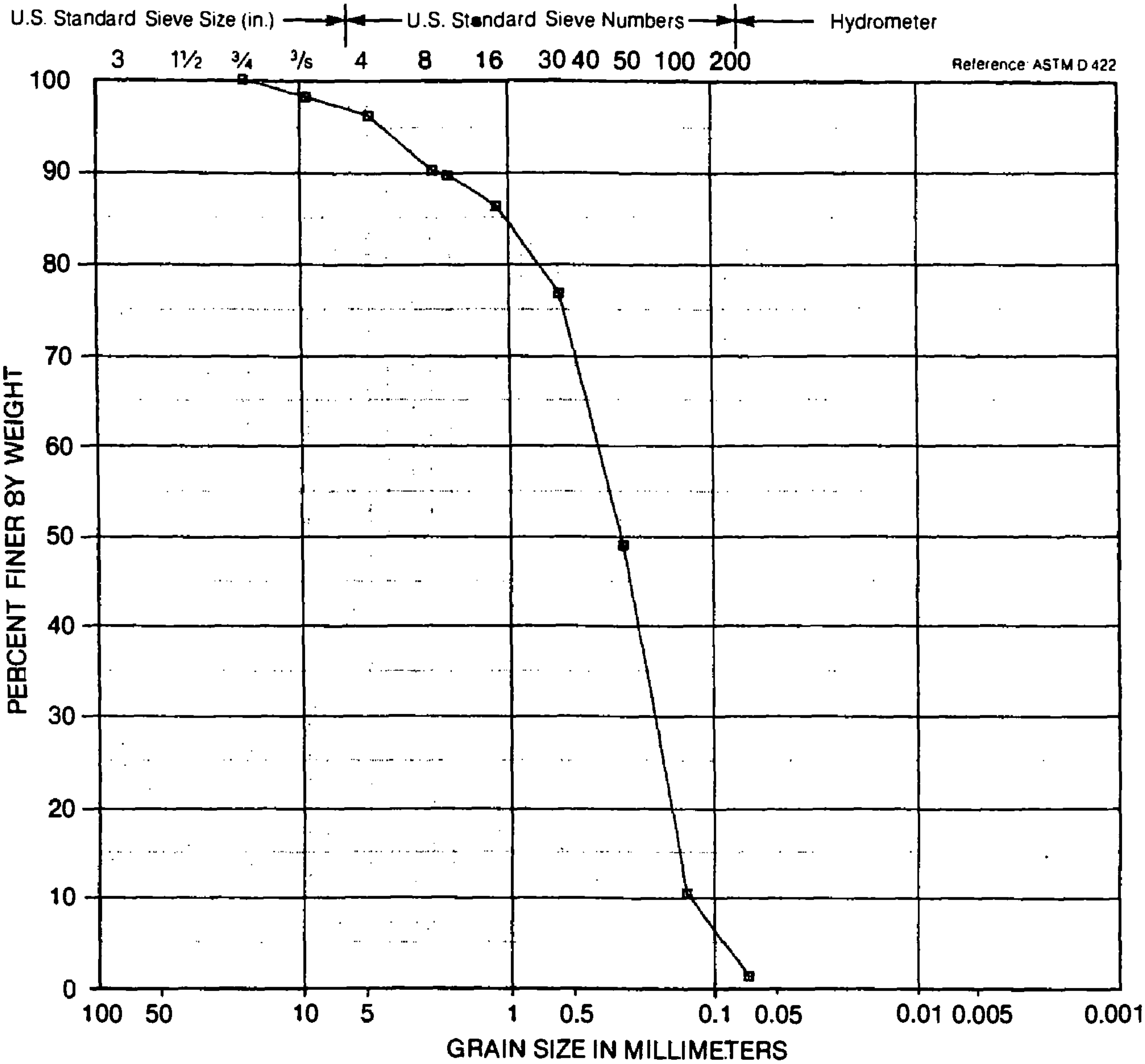
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
20

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	[illegible]	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

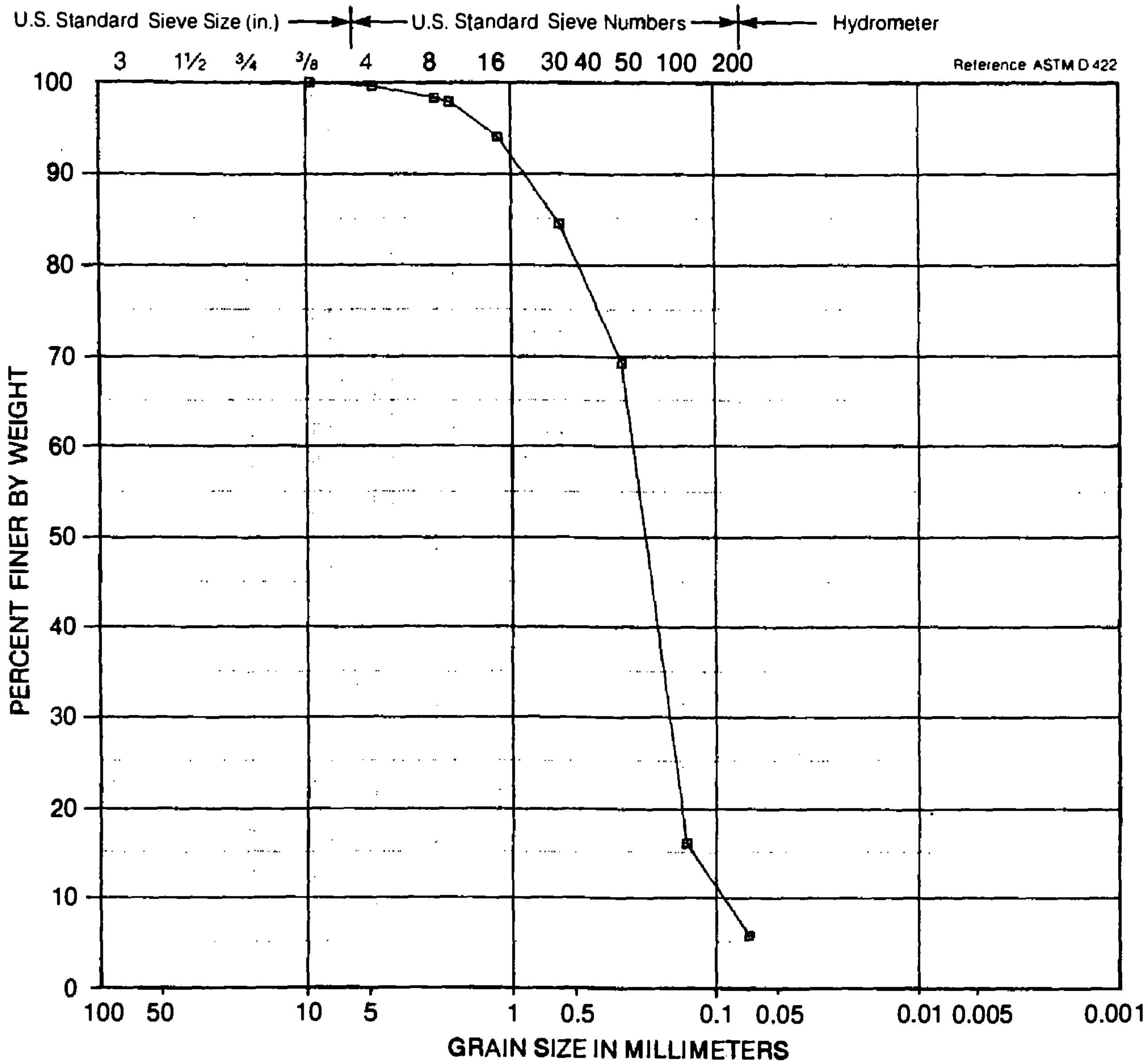
Symbol	Sample Source	Classification
▣	A-4 @ 10.5 FT	BROWN SAND (SP)

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
21

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	[illegible]	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

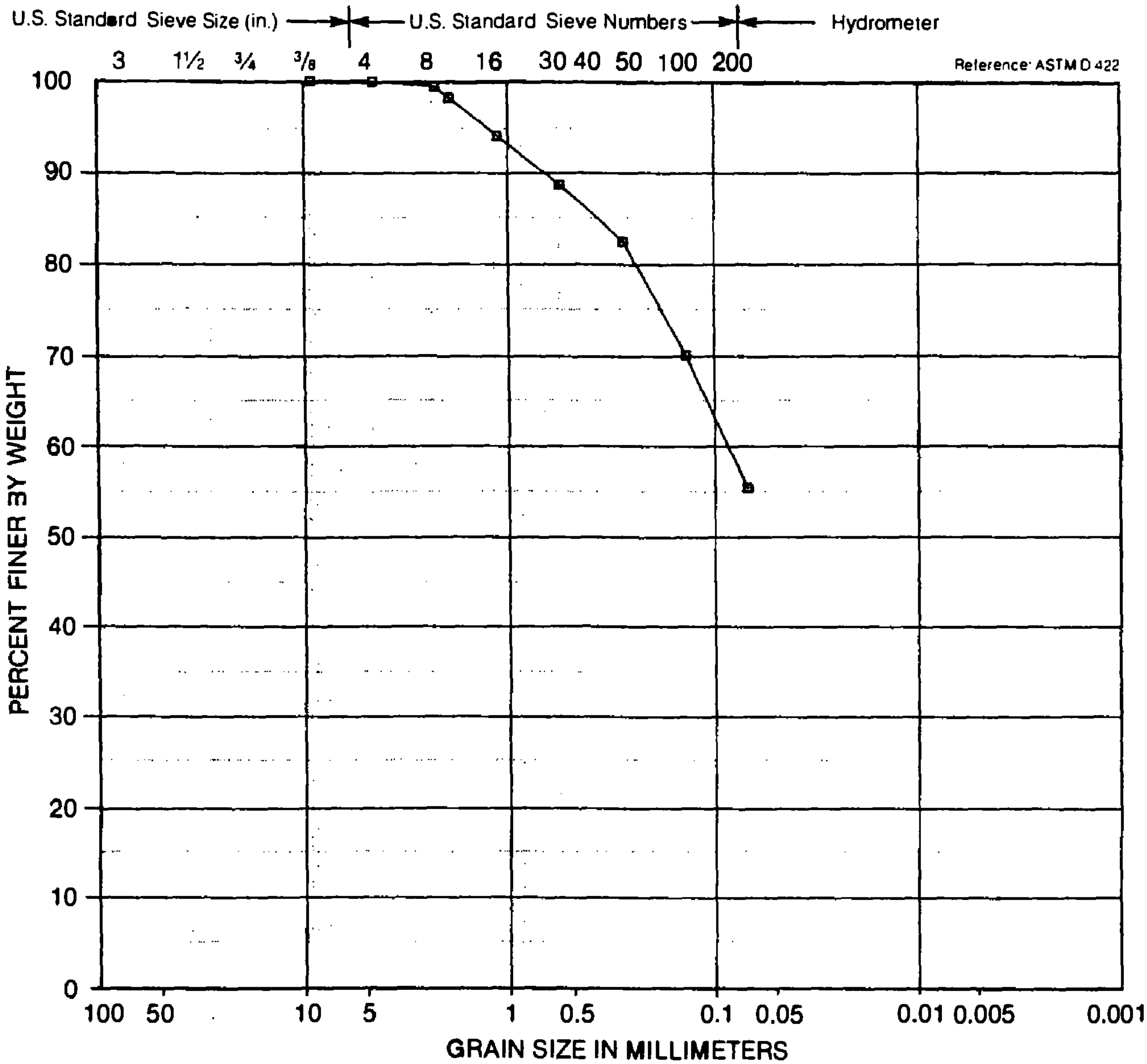
Symbol	Sample Source	Classification
▫	A-4 @ 11.0 FT	GRAY SAND W/ SILT (SP-SM)

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
22

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	BH	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▣	A-5 9 20.5 FT	GRAY SANDY SILT (ML)

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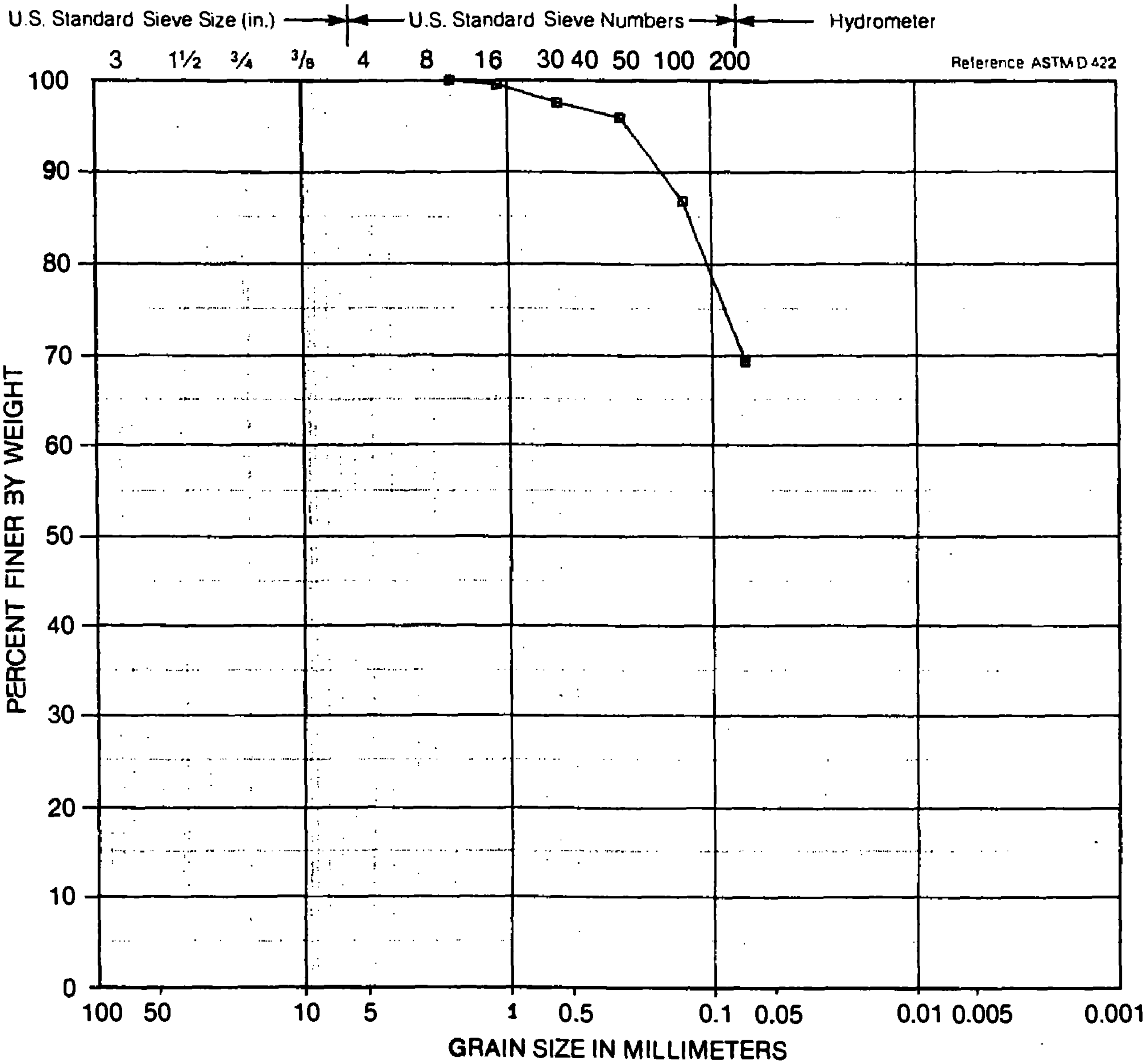
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
23

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	[illegible]	07-15-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▪	A-5 @ 45.7 FT	GRAY SANDY ELASTIC SILT (MH)

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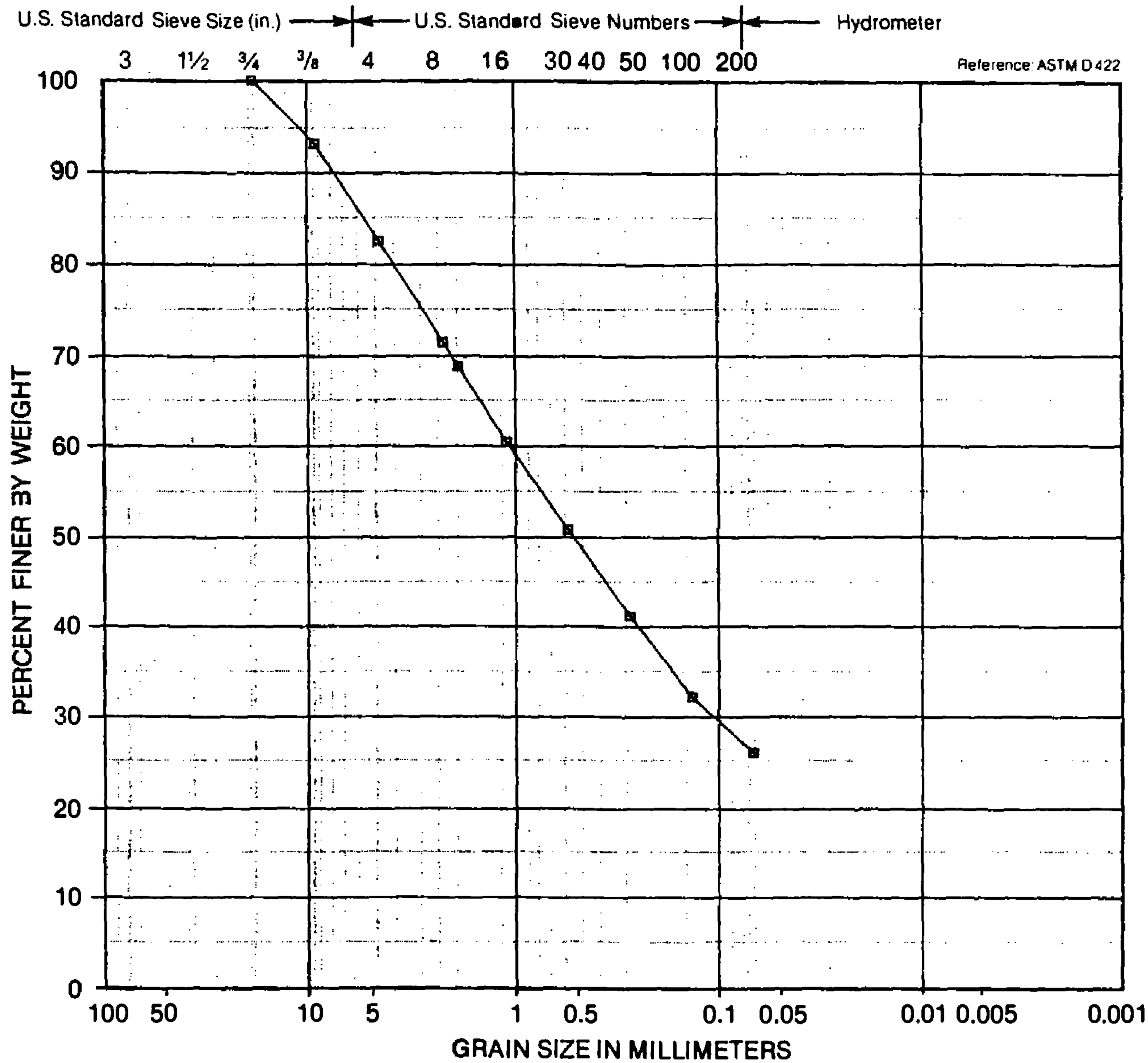
Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
24

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	RH	07-17-1991		11/91



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
▫	A-9 @ 9.0 FT	BROWN SILTY SAND W/ GRAVEL (SM)

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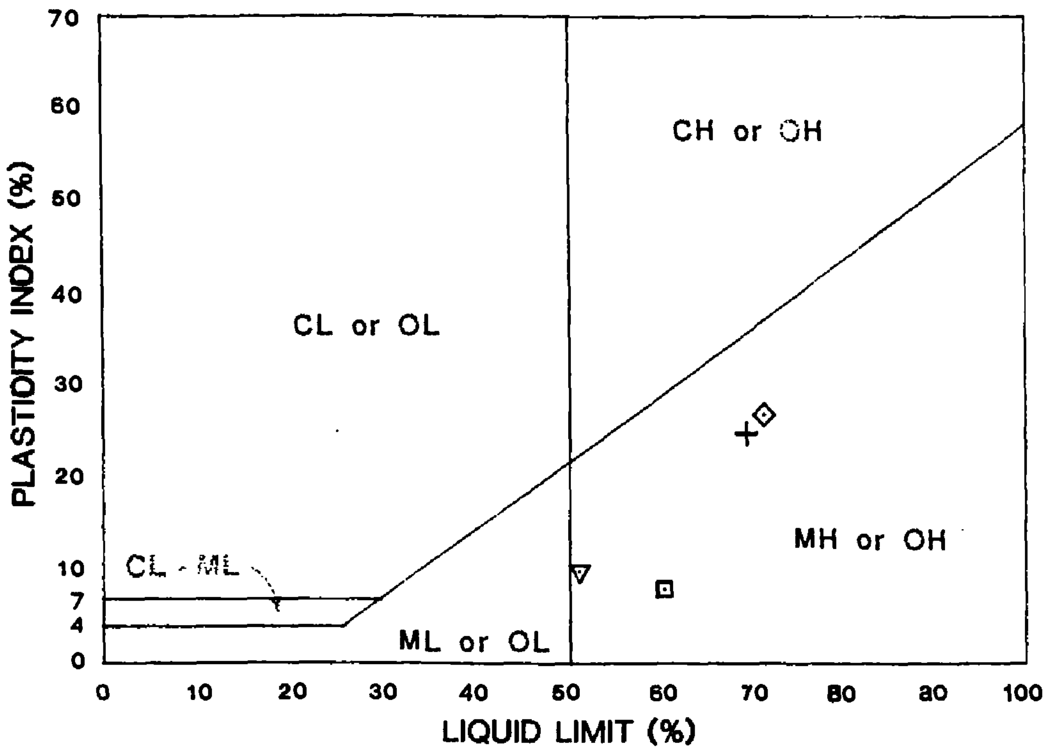
Engineers, Geologists
& Geophysicists

Particle Size Analysis
W.R. Grace Dam
Rainy Creek, Montana

PLATE
25

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	BH	07-15-1991		11/91



Reference: ASTM D-4318

SYMBOL	BORING NUMBER	DEPTH (feet)	CLASSIFICATION	LL (%)	PL (%)	PI (%)	MOISTURE CONTENT (%)
⊡	A-1	16.7	GRAY ELASTIC SILT W/ SAND (MH)	60	52	8	71.1
	A-1	35.5	OLIVE-GREEN SILTY SAND (SM)	NP*	NP	NP	
▽	A-2	25.2	OLIVE-GREEN ELASTIC SILT W/ SAND (MH)	51	41	10	63.2
◇	A-2	46.0	OLIVE-GREEN SANDY ELASTIC SILT (MH)	71	44	27	60.4
+	A-3	36.5	BLUE-GREEN ELASTIC SILT (MH)	69	44	25	64.0

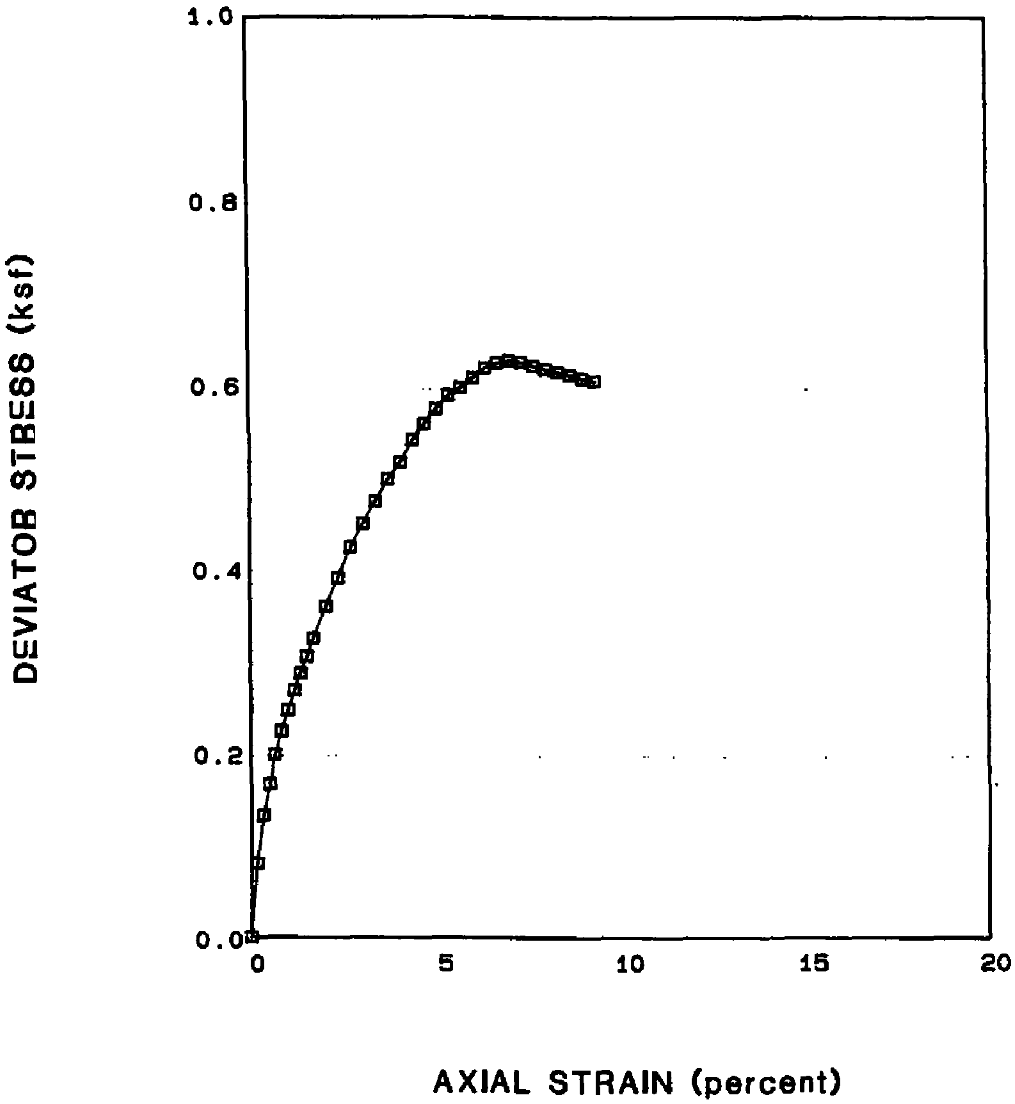
* NGN PLASTIC – NOT SHOWN ON CHART

HLA Harding Lawson Associates
Engineering and Environmental Services

Plasticity Chart
W.R. Grace Dam
Rainy Creek, Montana

PLATE **26**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5B91.053.03	[signature]	07-15-1991		11/91



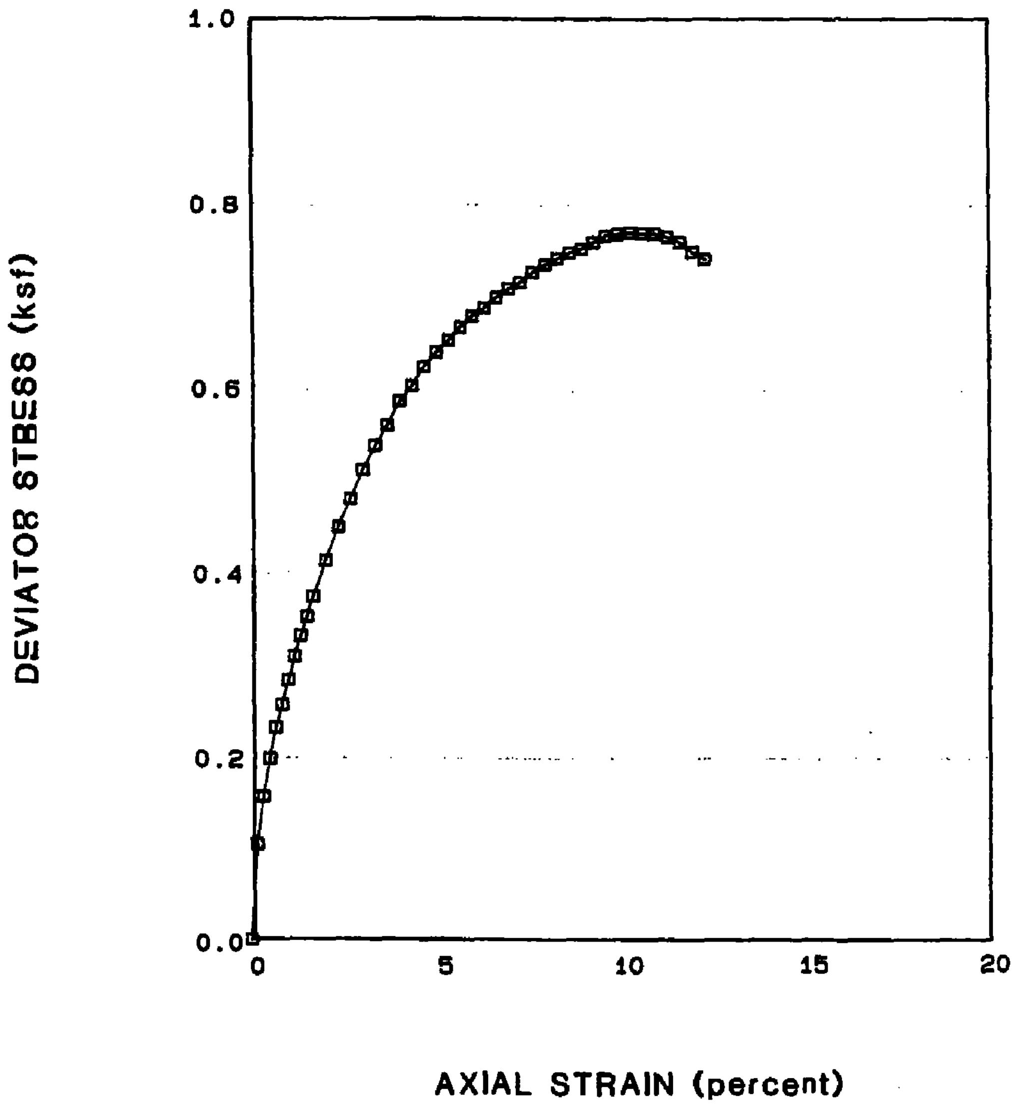
SPECIMEN TYPE UNDISTURBED		SHEAR STRENGTH	312	psf
DIAMETER (in) 2.67	HEIGHT (in) 6.00	STRAIN AT FAILURE	7.0	%
MOISTURE CONTENT 71.1 %		CONFINING PRESSURE	450	psf
DRY DENSITY 56 pcf		STRAIN RATE	0.60	%/min
CLASSIFICATION GRAY ELASTIC SILT W/ SAND (MH)			SOURCE A-1 @ 16.7'	

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Unconsolidated - Undrained
Triaxial Compression Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE
27

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	FH	07-12-1991		11/91



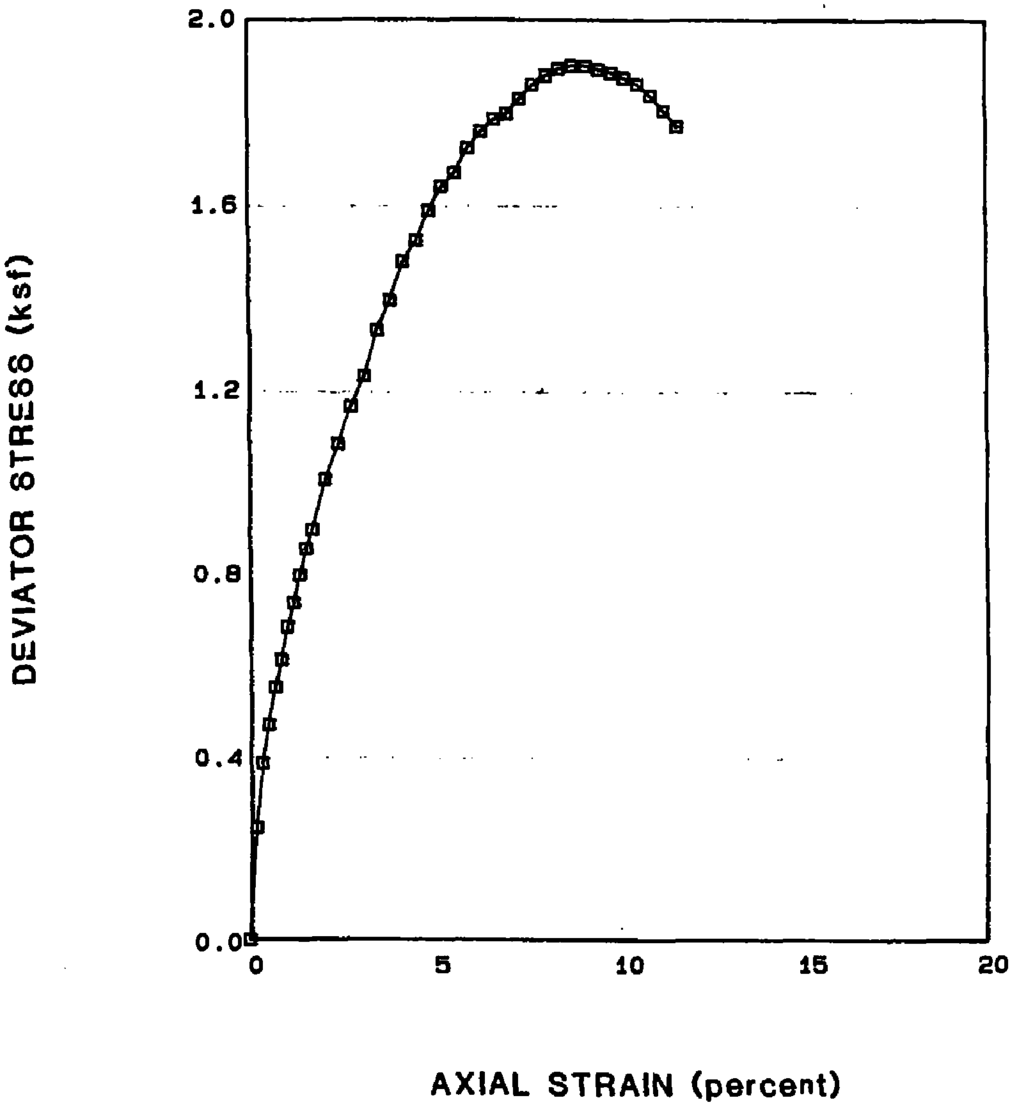
SPECIMEN TYPE	UNDISTURBED		SHEAR STRENGTH	382	psf
DIAMETER (in) 2.87	HEIGHT (in) 6.00		STRAIN AT FAILURE	10.3	%
MOISTURE CONTENT	63.2	%	CONFINING PRESSURE	500	psf
DRY DENSITY	62	pcf	STRAIN RATE	0.60	%/min
CLASSIFICATION	OLIVE-GREEN ELASTIC SILT W/ SAND (MH)			SOURCE A-S 9	25.2'

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Unconsolidated - Undrained
Triaxial Compression Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE
28

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.053.03	EH	07-12-1991		11/91



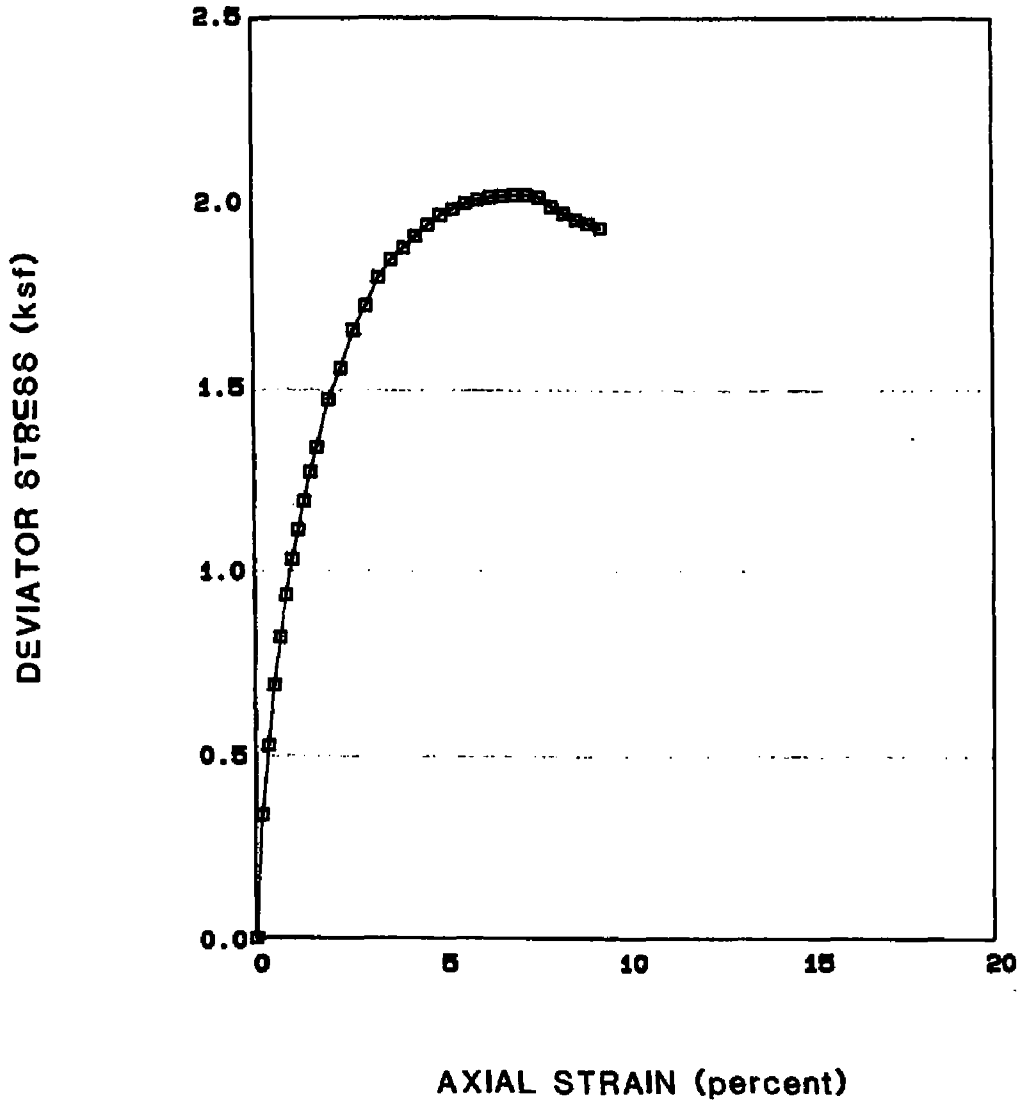
SPECIMEN TYPE UNDISTURBED		SHEAR STRENGTH 946	psf
DIAMETER (in) 2.87	HEIGHT (in) 5.70	STRAIN AT FAILURE 8.6	%
MOISTURE CONTENT 60.4	%	CONFINING PRESSURE 800	psf
DRY DENSITY 64	pcf	STRAIN RATE 0.60	%/min
CLASSIFICATION OLIVE-GREEN SANDY ELASTIC SILT (MH)		SOURCEA-8 @ 46.0'	

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Unconsolidated - Undrained
Triaxial Compression Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE
29

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5691.053.03	BH	07-12-1991		11/91



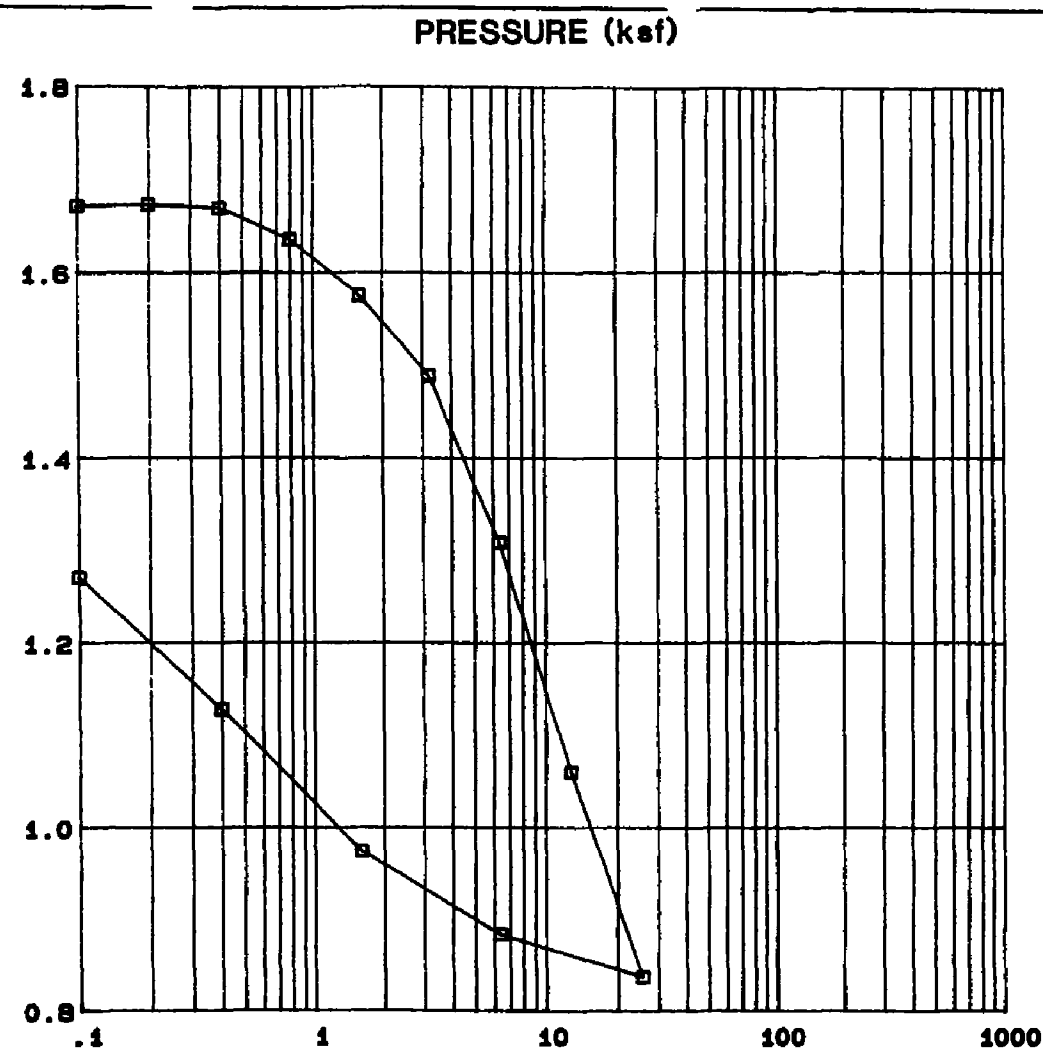
SPECIMEN TYPE UNDISTURBED		SHEAR STRENGTH 1005	psf
DIAMETER (in) 2.85	HEIGHT (in) 6.00	STRAIN AT FAILURE 7.3	%
MOISTURE CONTENT 54.0	%	CONFINING PRESSURE 750	psf
DRY DENSITY 62	pcf	STRAIN RATE 0.50	%/min
CLASSIFICATION BLUE-GREEN ELASTIC SILT (MH)		SOURCE A-S @ 36.8'	

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Unconsolidated - Undrained
Triaxial Compression Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE 30

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	8681.063.03	RH	07-12-1991		11/91



Reference: ASTM D-2435

SPECIMEN TYPE TRIMMED		BEFORE TEST			AFTER TEST	
DIAMETER (in) 2.43	HEIGHT (in) 0.00	MOISTURE CONTENT	w_o	57.1 %	w_f	43.0 %
OVERBURDEN PRESSURE, σ_{vo}' psf		VOID RATIO	e_o	1.68	e_f	1.27
PRECONSOL PRESSURE, $(\sigma_{vo}')_{max}$ 2800 psf		SATURATION	S_o	100 %	S_f	100 %
COMPRESSION INDEX, C_c 0.78		DRY DENSITY	γ_d	59 pcf	γ_d	61 pcf
LIQUID LIMIT —	PLASTIC LIMIT —	PLASTICITY INDEX —		SPECIFIC GRAVITY 2.08		
CLASSIFICATION OLIVE GREEN SANDY ELASTIC SILT (MH)		SOURCE A-2 @ 48.7 FT				

HLA Harding Lawson Associates
Engineers, Geologists & Geophysicists

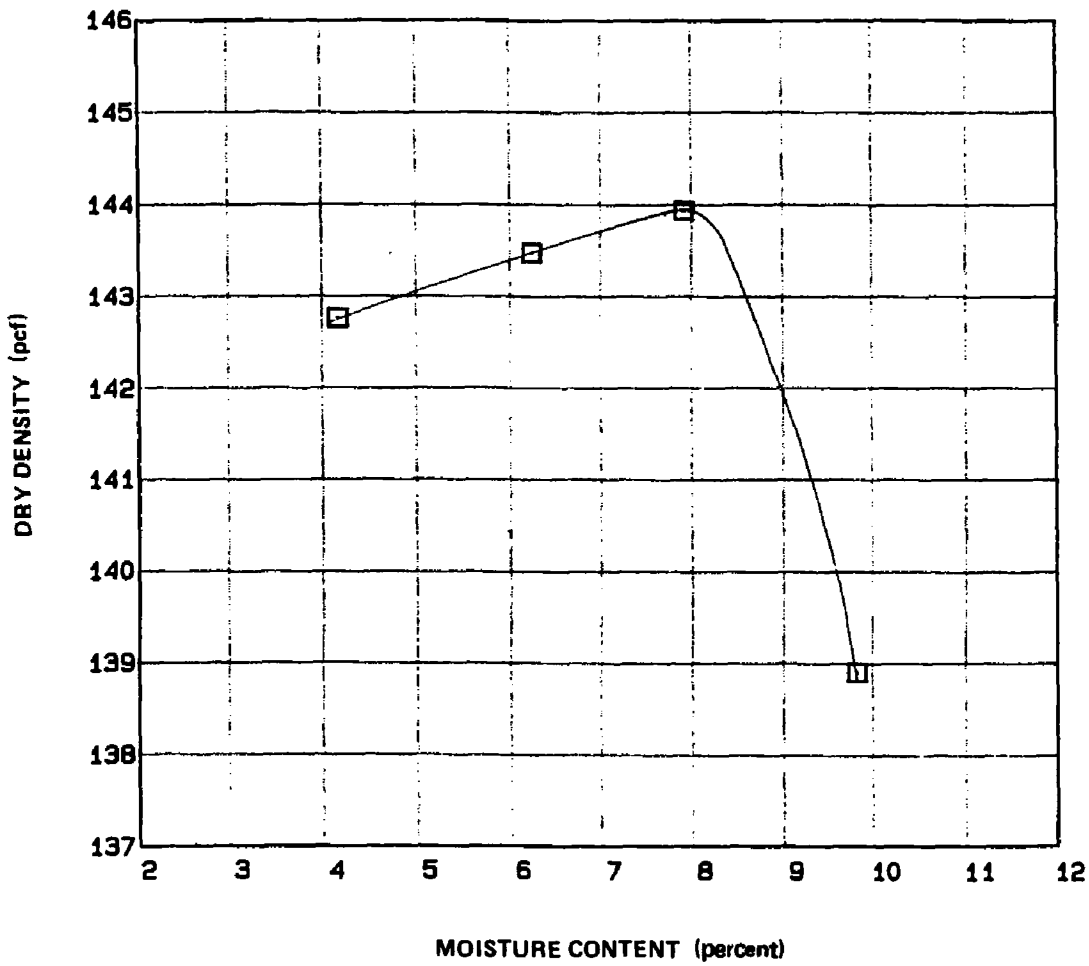
Consolidation Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE **31**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5891.063.03	EFH	07-24-1991		11/91

105649

MAXIMUM DRY DENSITY (pcf)	144
CORRECTED MAXIMUM DRY DENSITY (pcf)	144
OPTIMUM WATER CONTENT (%)	7.9



Reference: ASTM D-1557

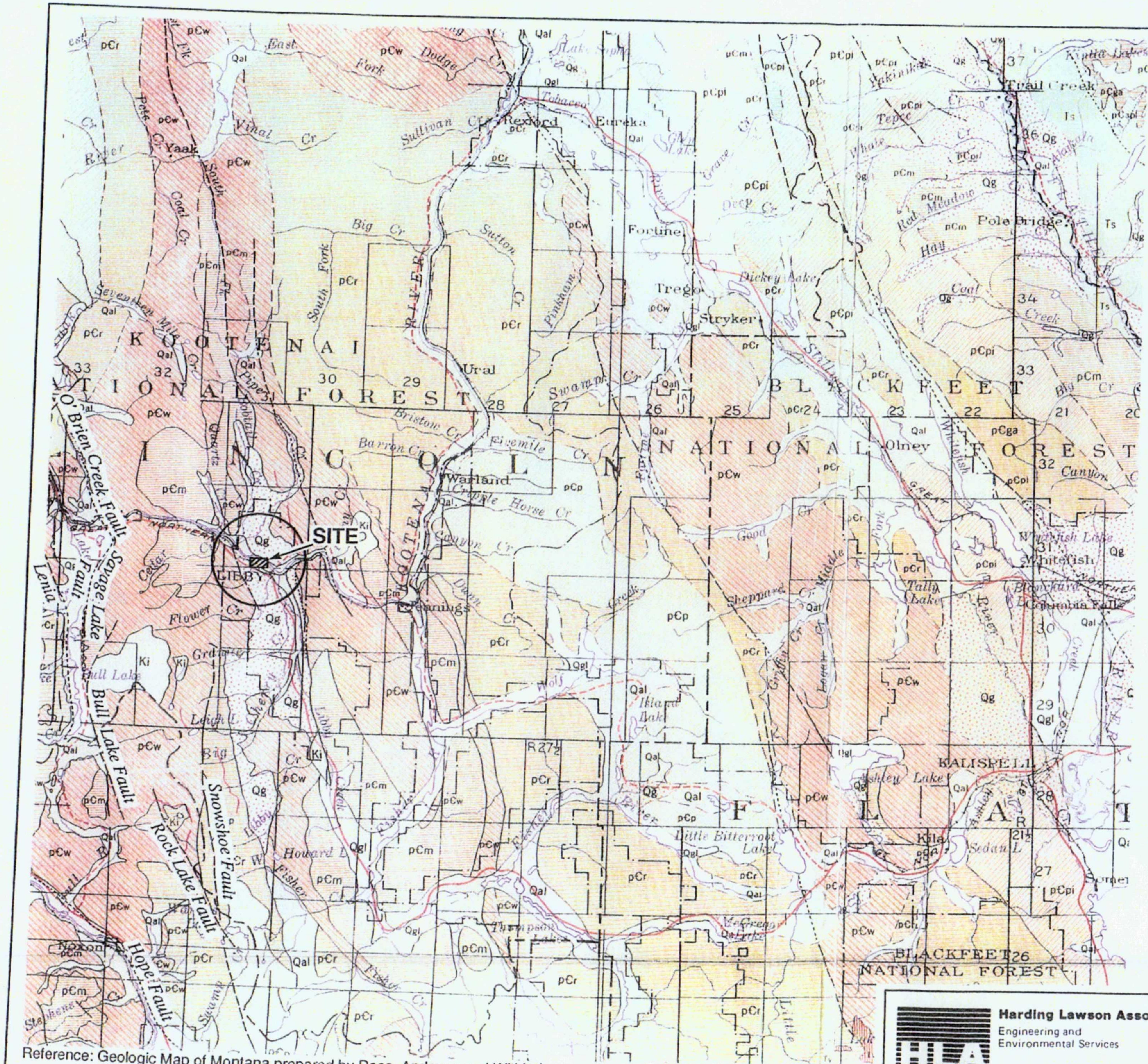
	1	2	3	4
MOISTURE CONTENT (%)	4.1	6.2	7.9	9.8
DRY DENSITY (pcf)	143	143	144	139
% PASSING ¾" 100.0	SPECIFIC GRAVITY (g/cc) 2.70		MOLD DIAMETER 6.00	
CLASSIFICATION BRN SAND W/ SILT G GRVL (SP-SM)			SOURCE BULK @ 0.0 FT	

HLA Harding Lawson Associates
Engineers and Geoscientists

Compaction Test Report
W.R. Grace Dam
Rainy Creek, Montana

PLATE
32

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	5691.053.03	PS	07-17-1991		11/91



EXPLANATION

Qal - Mainly valley fill consisting of silt, sand and gravel

Qg - Glacial drift; morainal and outwash plain deposits of mountain glaciers (Quaternary)

pCm - Missoula Group, chiefly red, maroon or purple argillite, sandy or quartzitic argillite, and generally impure quartzite and limestone (Pre-Cambrian)

pCw - Wallace Formation; dark gray argillite, arenaceous and argillaceous limestone and gray limy quartzite with shale and sandstone in large areas (Pre-Cambrian)

Geologic boundary
Dashed where approximately located

Geologic boundary indicated by the compilers on the basis of incomplete data

Fault, character not designated
Dashed where approximately located

Concealed fault

Thrust fault
T, upper plate

Fault indicated by the compilers on the basis of incomplete data

0 10 20 Miles

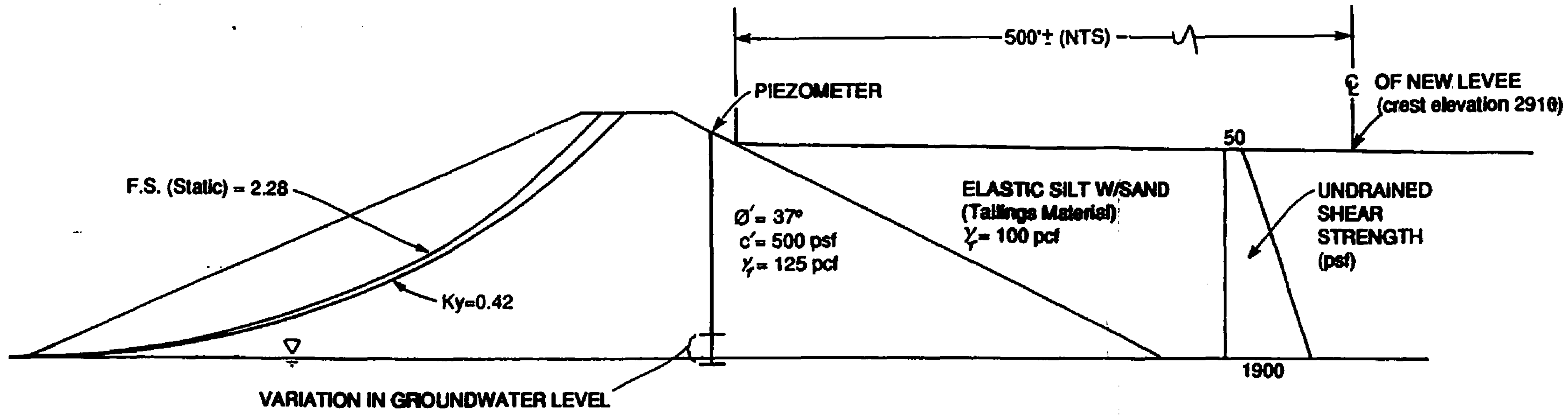
Scale (1:500,000)

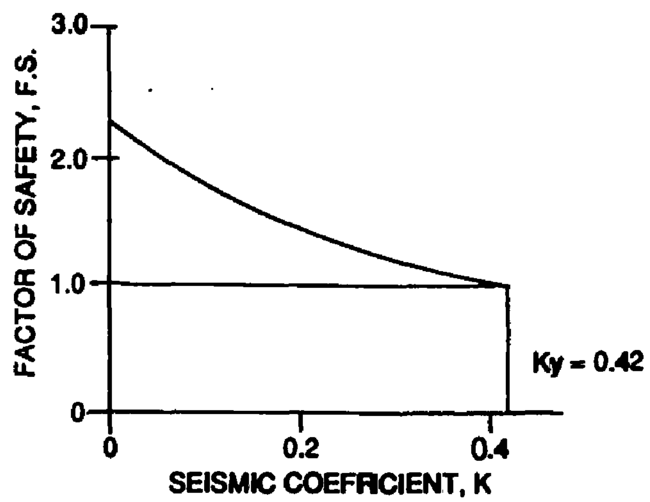
N

Reference: Geologic Map of Montana prepared by Ross, Andrews and Witkind (in cooperation with Montana Bureau of Mines and Geology), dated 1955.

135398

Harding Lawson Associates Engineering and Environmental Services		**Geologic Map of Site and Vicinity** W.R. Grace Dam Rainy Creek, Montana	PLATE **33**
DRAWN AM	JOB NUMBER 5891,053.03	APPROVED	DATE 2/92	REVISED DATE





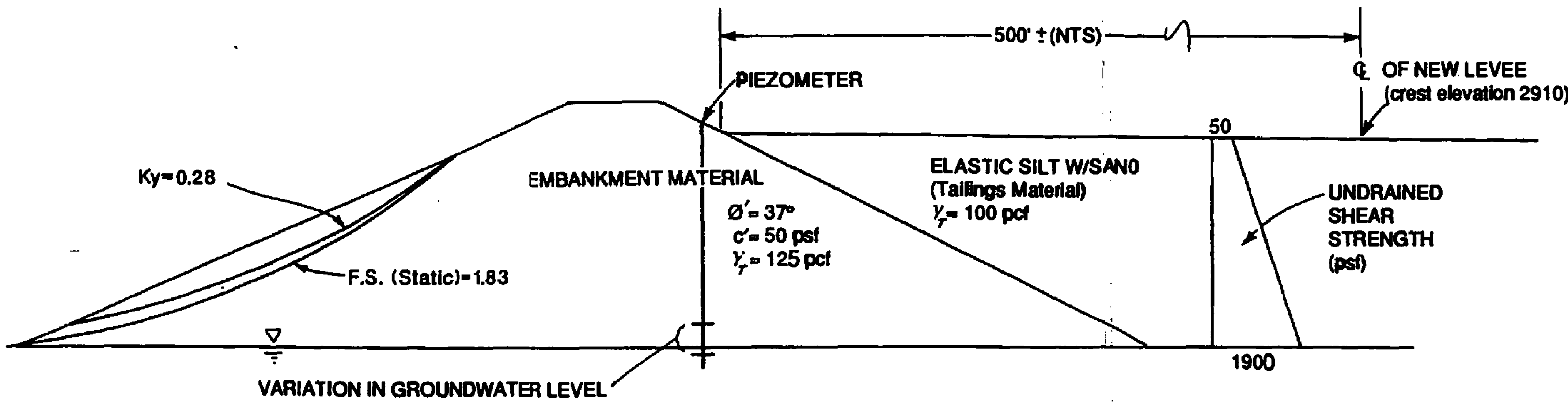
0 70
Scale in Feet

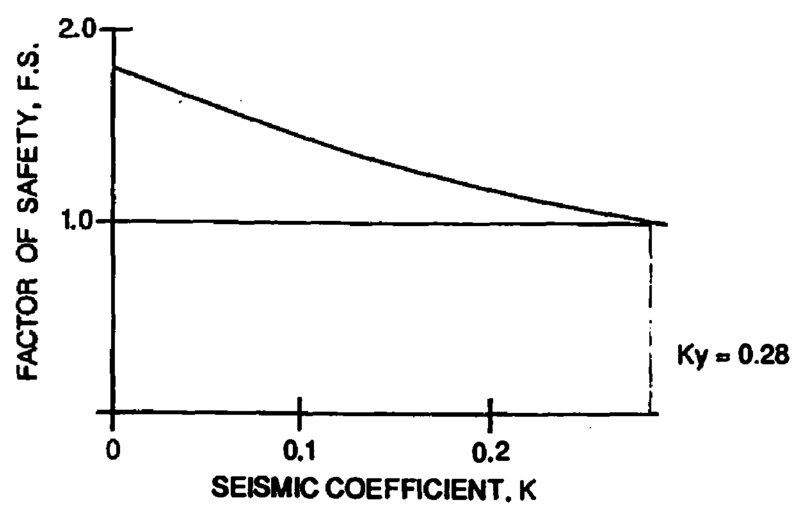
Case I, II: Stability Analysis - Water at 500 feet from Embankment (C' = 500 psf)

Sheet 1 of 2

HLA Harding Lawson Associates Engineering and Environmental Services		Analytical Model and Soil Properties W.R. Grace Dam Rainy Creek, Montana		PLATE 34a
DRAWN AM	JOB NUMBER 5891,053.03	APPROVED GLW	DATE 11/91	REVISED DATE

135398



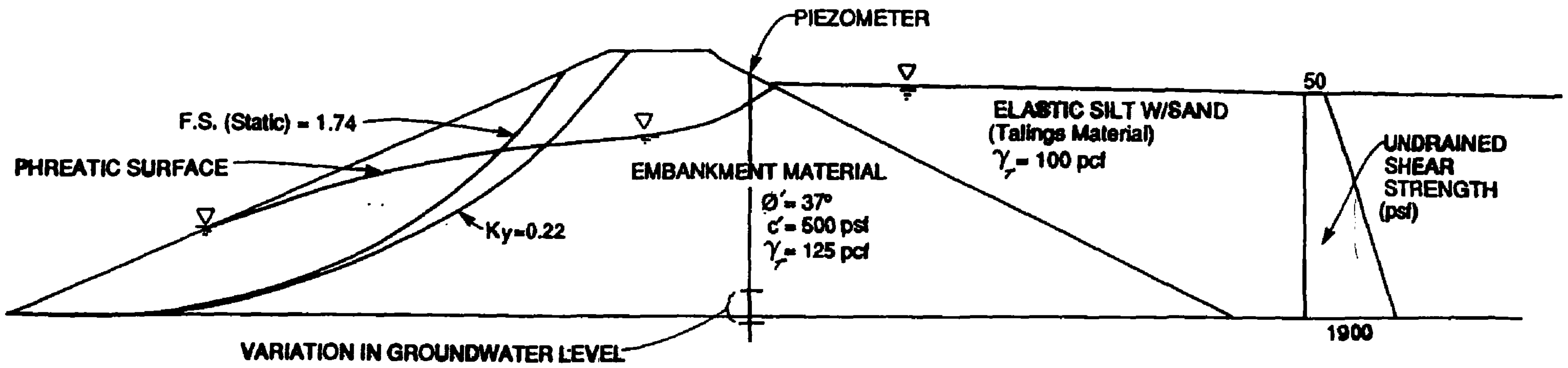


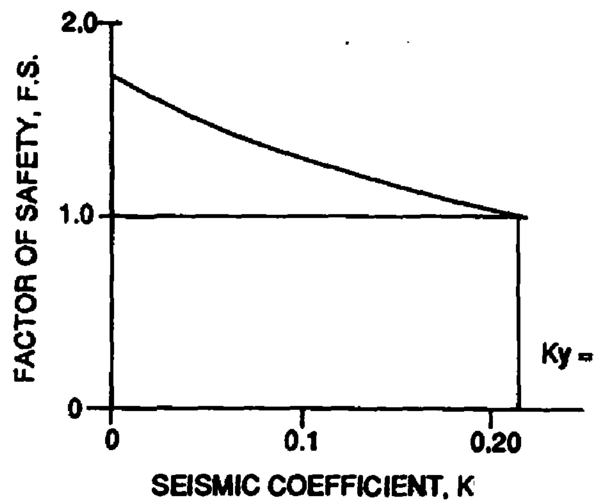
Case I, II: Stability Analysis - Water at 500 feet from Embankment (C' = 50 psf)

Sheet 2 of 2

HLA Harding Lawson Associates, Engineering and Environmental Services	Analytical Model and Soil Properties, W.R. Grace Dam, Rainy Creek, Montana		PLATE 34b
DRAWN: AM	JOB NUMBER: 5891.053.03	APPROVED: GLW	DATE: 11/91
			REVISED DATE

135398





0 70
Scale in Feet

Case III, IV: Stability Analysis - Water at Face of Embankment (C' = 500 psf)

Sheet 1 of 2

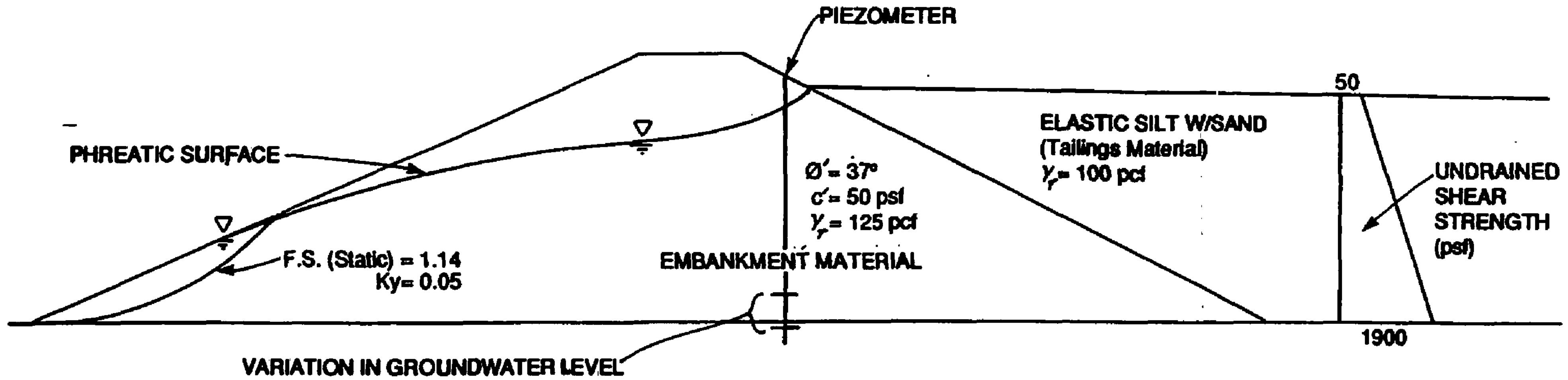
Harding Lawson Associates
Engineering and Environmental Services

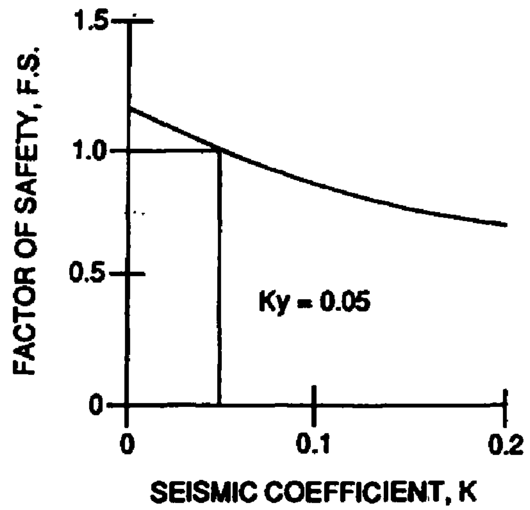
Analytical Model and Soil Properties
W.R. Grace Dam
Rainy Creek, Montana

PLATE
35a

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	

135398





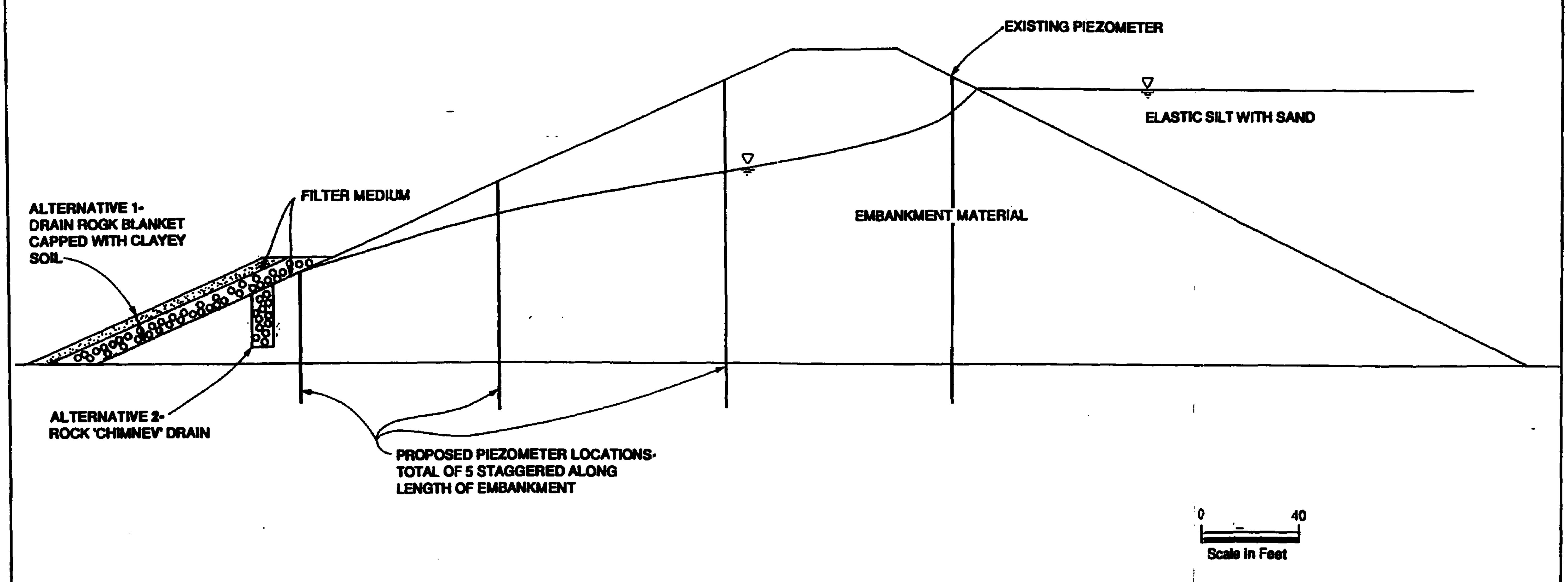
0 70
Scale in Feet

Case III, IV: Stability Analysis - Water at Face of Embankment (C' = 50 psf)

Sheet 2 of 2

Harding Lawson Associates Engineering and Environmental Services		Analytical Model and Soil Properties W.R. Grace Dam Rainy Creek, Montana		PLATE **35b**
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	

135398



Harding Lawson Associates
Engineering and
Environmental Services

Schematic Section of
Proposed Drainage/Monitoring System
W.R. Grace Dam
Rainy Creek, Montana

PLATE
36

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
AM	5891,053.03	GLW	11/91	

135396

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P.O. Box 609
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Attention: Mr. Alan Stringer

GLW/mfb/Bi2703-R71

QUALITY CONTROL REVIEWER

Donald Bruggers for
Keith H. Bergman
Geotechnical Engineer

A3-Reference 5

Schafer & Associates
P.O. Box 6186
Bozeman, MT 59715
(406) 587-3478

Waste Management
Land Reclamation
Resource Inventory
Agricultural Consulting

May 22, 1992

Mr. Alan R. Stringer
General Manager
Construction Products Division
W.R. Grace & Co.
P.O. Box 609
Libby, MT 59923-0609

RE: Responses to W.R. Grace Closure Plan - First Review

Dear Mr. Stringer:

The following are responses to questions regarding the tailings impoundment closure plan put forth in DSL's letter to you dated May 8, 1992 (Certified Mail No. P 676 686 599). They also incorporate some of the discussion on these items which took place in a meeting conducted in DSL's offices on May 19, 1992. Responses to Items 13 and 14 were provided by Harding Lawson Associates and relate to issues in their report on the stability of the tailings impoundment dam. All others relate to the conceptual flood routing design in the report prepared by Schafer and Associates.

Item 1. The percent of ground cover does not affect the runoff calculations for the PMF event, which is the primary concern for this site. The proposed system is designed to handle flows in excess of 0.5 PMF which is an order of magnitude greater than a 100 year event.

Changing the percent of ground cover from >75% to something less (60 to 70%) will have only a minor effect on the peak flows for 100 year events, which are expected to remain less than the designed control structure outflows.

Item 2. The excavated material referred to on page 4-13 of the Schafer and Associates report is for the emergency spillway construction only. Approximately 2,400 cubic yards of material will be excavated to construct the emergency spillway on the west abutment. The excavated material is proposed to be placed in the groin of the west abutment to provide additional protection. The material will be placed with standard earthmoving equipment and compacted. The existing dam face in this area will be prepared by stripping existing vegetation and keying new material into the existing materials in the dam face to improve stability. Should concerns over placement and stability remain, an alternate location will be selected to spoil the excavated material.

In the meeting with DSL on May 19th, it was determined that this question related to materials excavated from the outlet channel (hence, the reference to Plate 15). Materials excavated from this area will be used to construct the downhill slope of the outlet channel and to reinforce the area of the outlet channel where water is turned to cascade down to the valley floor.

Item 3. The decant tower pipeline will be plugged with concrete during closure. W.R. Grace has previously plugged a decant tower pipeline by pumping grout into the discharge end of the pipeline until the entire line is filled. This procedure was effective and will be used again.

Item 4. To maintain the water surface away from the face of the dam, it is necessary to construct the inlet channel as shown on Plate 8, with an inflow elevation of (approximately) 2904', and a gradient of 0.38%. To steepen the gradient would require a shorter channel which in turn would cause the water surface to be maintained closer to the face of the dam.

An alternative to lower the elevation of the control structure (within the dam) was reviewed and discarded due to the increased excavation required to construct the inflow channel, control structure, and outflow channel.

Sedimentation of material in the inlet channel is not anticipated to pose a significant problem. Initially, the majority of stream sedimentation is expected to occur in the wetland system upstream of the inflow channel where flow velocities are negligible. Should a minor amount of material settle in the inlet channel during periods of low flows, increased flow velocities during peak events will most likely clean the channel because these will be relatively fine materials. Over the years, the wetland may begin to accumulate sediment at the upper end of the impoundment (See also Items 11 and 15). Under these conditions the sedimentation may, over geologic time, extend into the upper portion of inlet channel but the result will be for the stream to establish a naturally stable gradient in materials washed out of Upper Rainy Creek and Fleetwood Creek leading to the outlet structure at 2900' through the constructed inlet channel.

The model used to calculate the inlet channel velocity of 5.5 fps is located toward the end of Appendix D.

Item 5. Fleetwood Creek will be restored to a channel constructed in natural material adjacent to the toe of the coarse tailings dump. This channel will be built to handle a 100 year storm event in accordance with DSL's reclamation guidelines. We anticipate using existing streambed materials, which are quite coarse materials, to armbr the toe of the coarse tailings. Two or three small ponds currently exist along this stretch of Fleetwood Creek. We would leave these ponds as they are. Details of the channel will be forthcoming with the final design.

Item 6. We do not anticipate that infiltration from a localized area near the toe of the impoundment would produce significant stability problems for the dam. However, an impermeable channel liner constructed of either HDPE or clay, ~~extending to a suitable distance from the face of the dam~~ (approximately 500 feet) could be incorporated into the final design. W.R. Grace is amenable to this as an element of the final inlet channel design.

Item 7. The interface between the inlet channel and the control structure will be heavily armored with a layer of rock rip-rap. We also intend to incorporate design features in the outlet structure to protect against the possibility of channeling between the structure itself and the soil materials next to it. Details will be forthcoming with the final design.

Item 8. Following excavation, the control structure will be placed, properly bedded, and the excavation backfilled with the original material. The backfill will be placed in proper lifts (12" maximum) and compacted to a density that meets or exceeds the original specifications used during the construction of the dam. A primary concem for the design of the outlet structure will be to prptect against differential settlement. We will need to characterize the underlying materials carefully to do this. Ideally, it will be possible to construct the outlet control structure in such a way that it rests entirely on bedrock. If this is not possible, a suitable design can be made with the structure resting entirely on the compacted dam materials. Details, sections, and specifications will be provided with the final design.

Item 9. The drop structures will be constructed (and anchored) to the specifications recommended by the SCS. This may include keying into bedrock, doweling, or other methods deemed necessary to provide a long-term stmcture. Details, sections, and specifications will be provided with the final design.

Item 10. The outlet channel will he designed for long-term geotechnical stability and we will give serious attention to the concem addressed by this question during final design. Our intent is to utilize bedrock foundation to the extent possible without creating unnecessarily large sidehill cuts. The bottom of the channel will be slightly sloped to the inside of the cut to keep the normal stream flow on bedrock materials thereby preventing infiltration losses.

It is not uncommon to constmct sidehill access roads on soil materials. The existing roadway which can be seen on Plate 15 (but is not identified as a road) is constructed on soils. The new road will not receive heavy traffic and is amply protected against erosion by the large channel between it and the sidehill. Also, excavated materials will be placed on the sidehill in benches which are keyed into existing materials in order to promote stability. These are factors which will help to maintain its integrity over the years.

Item 11. It is not likely that the impoundment will ever fill entirely with sediment. The outlet at 2900' elevation will be maintained as the only outlet for stream flow. Because of this, as sedimentation begins to fill the wetland area, the tendency will be for Rainy Creek to cut a natural stream channel in the sediment materials at the upper end of the impoundment and flow into the inlet channel which we are constmcting in the tailings materials. The question raised in Item 4 and our response to it are pertinent to this situation.

Also, we do not feel that a fiill PMF design criteria is apprqpriate. For example, in the Record of Decision for the impoundment dams at the Wann Springs Pond Operable Unit for the Silver Bow Creek/Butte Area NPL Site, the dam is designed and permitted for a 0.5 PMF event. We consider this to be an even more critical situation since this impoundment contains materials which are classified as hazardous. The criteria for this facility design were apparently set by EPA and DNRC. Comments by the Dam Safety Bureau on this issue would be appropriate.

Item 12. A system to control trash and debris will be included in the final design. It is anticipated that the general design approach will incorporate two sets of "pile fences" set across the inlet channel, one being immediately upstream of the inlet to the control stmcture and the other being a distance upstream. The pile fences are expected to intercept and divert the larger debris, while allowing the smaller pieces to pass. The control stmcture and drop stmctures are expected to pass the smaller pieces without problems arising.

It is not recommended to install a grate or other "smaller" debris capturing device over the entrance to the control structure. These tend to plug easily and are difficult (or impossible when inundated) to clean.

Debris collection devices are not considered necessaiy for the drop stmctures, and are not proposed.

Maintenance of the debris control devices will be part of the proposed maintenance plan.

Item 13. We concur with DSL's conclusion that the actual distance of the O'Brien Creek fauh from the mine site is 13 miles (21 km) and not 13 km. We also agree that this error in our analysis resulted in a conservative estimate of ground motion associated with a magnitude 7 event on the O'Brien Creek fault. We will correct this error and revise the report accordingly.

With regard to the potential for seismic activity on the Rainy Creek fault, according to our conversation with Mr. Mike Stickney of the Montana Bureau of Mines and Geology (MBMG), there is insufficient evidence of either Holocene (last 11,000 years) or Quaternary (last 2 to 3 million years) activity on this fault in our seismic analysis. Our seismic design criteria considers the occurrence of a magnitude 5.5 event at a very close distance to the site, resulting in a peak ground acceleration (PGA) of 0.30 gravity (g). We believe that a PGA of 0.30g conservatwely envelopes expected seismic activity at this site under the presently known tectonic framework.

Item 14. We propose to monitor the location of the phreatic surface within the embankment by installing six additional piezometers in the downstream face of the dam. Three piezometers will be installed near the toe of the dam, with one lined-up against the existing piezometer P-2 (please refer to the Harding Lawson Associates report for the location of the six existing piezometers); two piezometers will be installed near the existing drainage pipe to monitor possible long-term leakage from this system; and one piezometer will be installed near the proposed outlet channel. We judge that a periodic monitoring of these twelve piezometers will enable us to detect any change in the phreatic surface. Remedial actions such as those presented in our geotechnical report dated Febmary 3, 1992 will be taken, should the phreatic surface approach unacceptable levels.

Item 15. Again, we do not feel that a full PMF is appropriate as a design basis within the criteria set forth by the Dam Safety Bureau (See Item 11). A spillway to accommodate a full PMF is technically feasible; however, it would entail large sidehill cuts on the west side of the tailings dam and relocation of the Forest Service road.

If we can be of further assistance in addressing these issues, please call.

Sincerely,

T. J. Hudson

Thomas J. Hudson
Project Manager

A3-Reference 6

GRACE

onstruction Products Division

TO: R. M. Vining

FROM: J. W. Wolter

cc: O. M. Favorito
W. J. McCaig - Libby
R. E. Schneider
E. S. Wood

DATE: December 1, 1981

SUBJECT: Libby Impoundment Dam

Under the authority of Public Law 92-367, an engineering firm, Morrison-Maierle, Inc., inspected our impoundment dam on July 25, 1980 and again on August 13, 1981. A report titled "Rainy Creek Basin - Zonolite Tailings Dam" was issued by Morrison-Maierle in September 1981 and approved by the Corps of Engineers, Department of the Army.

The comprehensive engineering report was based on events resulting from a "100-year flood". This hypothetical event is considered as the probable maximum flood (PMF) and is the resultant from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF is based on a July to August thunderstorm that produces 6" of rain in one hour and 8" of rain in six hours. This flood is also based on a 100-year, 24-hour antecedent storm of 3.4" as outlined from the National Oceanic and Atmospheric Administration (NOAA). This antecedent storm produced .3" of run-off prior to the PMF storm. Further conservatism assumes that the diversion structure on Rainy Creek had failed, allowing the entire run-off from Rainy Creek to join Fleetwood Creek and enter the reservoir.

This combination of events would result in the dam overtopping during the PMF when approximately 55% total flood volume enters the reservoir. The flood subsequently would occur downstream as a result of overtopping of the dam. The aforementioned PMF produces an estimated 3,770 acre feet of water. Since the reservoir is 68.5 acres in size and contains less than five (5) feet of water, the total water normally behind the dam is less than nine percent of postulated PMF.

This PMF essentially generates 43,400 cubic feet per second (cfs) of flow from the 9.7 sq. mile basin. To put this in perspective, the average uncontrolled water flow in the Kootenai River for 52 years (prior to construction of the Libby dam) on record is for July at 30,000 cfs, August for 13,000 cfs and September at 17,000 cfs. Or looking at it in another way, this flood would be 45% greater than the average uncontrolled flow during the month of July in the Kootenai River. The maximum flow of water on record at Libby in the Kootenai River was 121,000 cfs in the year 1916. This flow was generated from 9,900 sq. miles of drainage in the U.S. and Canada.

Findings and conclusions included in the report which is based on criteria developed for the U.S. Army Corps of Engineers under their "guidelines for safety inspection of dams" results in the dam being classified as Category I, having a "high downstream hazard potential". Their findings state "the dam is located such that its failure could endanger more than a few lives and cause excessive economic loss".

We disagree with these findings and have included in our objections to the Corps a letter from Lyle Lewis, a professional engineer with the engineering firm of Harding and Lawson Associates, stating the following: "It is our opinion that the downstream hazard should be classified as low or moderate". Further statements from Mr. Lewis include: "For three miles below the dam, in Rainy Creek Valley, there is nothing but the oiled roadway".

In a draft document sent to the Mine Safety and Health Administration (MSHA), indications were given that "failure could endanger may lives and cause excessive economic loss". This draft document resulted in a critical response from MSHA and is included as an exhibit in the Corps of Engineers final report. In order to mitigate this unreasonable interpretation, the Corps of Engineers has prepared a memo for MSHA indicating that the only potential damage referred to in the report was to the facilities owned by Grace and that an acceptable monitoring and evacuation plan of the Grace facilities would prevent severe losses or risks to Grace personnel. A copy of this clarification memo from Colonel Leon K. Morashi, the District Engineer for the Corps of Engineers, is attached as Exhibit 2.

Recommendations from this report and our implementation plans are as follows:

1. Immediately develop, implement and periodically test an emergency warning plan for use in the event of impending embankment overtopping for structural failure.

   This emergency warning plan is being implemented. A copy of the first draft of this plan is attached as Exhibit 1.

2. Periodically test the decant line in the section which passes through the embankment for possible leaks which could threaten the embankment.

   This procedure has been in effect and we have retained the services of Harding and Lawson as our consultant engineering firm for both construction and maintenance on this facility.

3. Conduct more detailed hydrologic and hydraulic routings to better determine the downstream hazard potential and to establish a safe minimum flood storage volume and spill requirement.

   Lyle Lewis, consulting engineer for Harding and Lawson, has been given the assignment to work with Libby Engineering and develop the details requested.

4. Continue to conduct inspections of the dam on an annual basis by engineers experienced in dam design and construction, continue to monitor and evaluate piezometers, foundations, toe drains and maintain construction log of all additions and modifications to the project.

   This procedure has been in effect and utilizes the engineering services of Harding and Lawson.

The PMF as stipulated by the Corps of Engineers utilizing their computer program HEC-1 would result in the dam overtopping after 55% of the flood water entered the ponded area. Since the probable maximum precipitation (PMP) would generate a peak flow of 43,400 cfs and a volume of 3,770 acre feet, the present spillway is inadequate to withstand this surge. The resultant would possibly lead to a breach in the dam that could wash up to 125,000 cu. yds. of dam, slimes and coarse tailings from the impoundment area. This is about 6.6% of the total 1,880,000 cu. yds. of material presently stored above the dam. While the majority of this material will form a delta immedlately below the dam, the slimes and finest fractions would continue down stream. It is postulated that 25,000-30,000 cu. yds. could reach the Kootenai River or about 1.5% of the stored material.

Harding and Lawson concurs with our local management that an antecedent storm followed by a 6" storm in one hour is unreasonably conservative for the Libby area. We will, therefore, attempt to pursuade the Corps of Engineers to revise the PMF based on reliable meteorologic and hydrologic data. This data will be gathered and presented to the Corps of Engineers by Harding and Lawson under the supervision of CPD.

In the event that Harding and Lawson determines, after thoroughly evaluating hydrologic and meteorologic data, that conditions leading to a PMF are realistic or if they are unable to convince the Corps of Engineers that conditions leading to a PMF are unreasonable, the following actions will be taken:

1. A decision will be made to determine the most economic and technologically safe method to control a PMF.

2. The Corps of Engineers will be consulted to their concurrence as to the method or methods recommended.

3. A schedule for implementation will be presented to CPD and Grace management for spending authorization.

At this time it would appear that a concrete spillway costing approximately $150,000 would adequately handle a PMF. Alternative controls considered for a PMF included:

1. Constructing an aquaduct capable of diverting all water around the dam and costing $2.4 million.

2. Maintaining the dam at levels capable of holding the full 3,770 acre feet of water behind the dam at a cost of $1.7 million.

If the Harding and Lawson studies support a lesser PMF (and the Corps of Engineers concurs), it is likely that the existing spillway will prove adequate or, at most, a modestly improved spillway will be sufficient at costs of less than $20,000.

Jack

Jack W. Wolter

JWW:dlc
Attachments

A3-Reference 7

DEPARTMENT OF NATURAL RESOURCES AND CONSERVATION

WATER RESOURCES DIVISION • 1424 Ninth Avenue, Helena, MT 59601
(406) 444-6601 Telefax (406) 444-0533

BRIAN SCHWEITZER, GOVERNOR

STATE OF MONTANA

DIRECTOR'S OFFICE (406) 444-2074
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FO BOX 201601
HELENA, MONTANA 59620-1601

MEMO

To: Tim Davis, WRD Administrator; Laurence Siroky, WOB Bureau Chief

From: Michele Lemieux, P.E. Montana Dam Safety Program Manager

Date: Tuesday, February 01, 2011

Re: Seismic Hazard Assessment – Flower Creek Dam

The purpose of this memo is to provide a cursory assessment of the seismic hazard at Flower Creek Dam. This assessment includes an estimate of ground shaking that could occur from hypothetical earthquakes in the area, as well as a determination of the probability of these various seismic events occurring.

<u>Estimate of Ground Shaking from Hypothetical Earthquakes</u>

Figure #1 shows distribution and magnitude of recent earthquakes and mapped faults in the vicinity of Libby, Mt (Source: Wong, et al, 2005). Figure #1 also shows the estimated peak ground acceleration at the base of Flower Creek Dam from various hypothetical events calculated using the DNRC MTShake Ground Shaking Estimation Program (Wong, 2008). Peak ground acceleration (PGA) is a seismic parameter commonly used to assess severity of ground shaking. PGA is reported as a fraction of the gravitational constant g. For reference, 0.01g could cause broken dishes and windows; 0.1Sg can cause heavy furniture to be overturned; 0.32g can cause buildings to shift off of their foundation while 0.7g can destroy well built wooden structures (Richter, 1958). Hypothetical events were divided into two categories: Those that may occur along regional mapped faults with geologically recent activity and those on unmapped or "blind faults" in the vicinity of Libby.

Large events on the Savage Lake, Bull Lake, Swan or Nyack faults cause only minimal ground shaking at Flower Creek Dam (PGA = 0.08g). To verify results calculated by the MTShake program, a check was run by Ivan Wong of URS Corporation using the latest methodology for the closest fault (Savage Lake Fault, 18km distance). The average peak ground acceleration at the dam site using the new methods was calculated to be 0.11g. The URS calculations and related technical information are included in Appendix A.

Not all faults are mapped. Glaciation during the last ice age and dense forest cover tends to obscure surface manifestations of fault movement. Conventional thinking is that blind fault earthquakes do not have magnitudes that exceed 6.5. The logic behind this assumption is that if a fault is capable of producing a magnitude 7 or greater earthquake, there is generally some surface manifestation of the fault. That said, it is possible that unmapped faults capable of generating a magnitude 7 or greater earthquake could be present. The magnitude 7.0 earthquake in Haiti occurred on a fault which had minimal surface expression. Assuming that the maximum magnitude of a blind fault earthquake in close proximity to the dam is 6.5, the peak horizontal acceleration at the dam would be 0.23g.

For reference, the DNRC seismic response procedure (DNRC, 2008) takes seriously any event over 0.2g. Immediate inspection is required for any dam that experiences ground accelerations at this level. Damage (cracking, foundation movement) can occur. However, historically, much higher accelerations are needed to cause catastrophic failure of a dam.

As discussed in the recent coring report on Flower Creek Dam (NTL Engineering, 2010), the poor condition of the concrete could result in damage and/or failure at accelerations of 0.16g or higher. *Thus, it can be concluded that a magnitude 6.5 earthquake on a blind fault in the vicinity of Libby could cause ground shaking sufficient to damage or fail Flower Creek dam.* Note that it is typical for accelerations at the top of the dam to be significantly amplified as compared to the dam foundation. Evaluating the amplification effects of a structure requires a complex analysis.

Seismic Event Probability

Figure#2 illustrates the 2% probability of exceedance in 50 years for the peak ground acceleration in the Libby area (Wong et al, 2005). This is equivalent to a 2500 year return period. The range in acceleration values in the Libby area is partially attributed to the presence of alluvium along the major river valleys, which can amplify ground motions. Flower Creek dam is located on Precambrian bedrock. Bedrock does not amplify ground motions in the same manner as alluvium. Available data indicate that there is a 2% probability in 50 years that Flower Creek Dam will see a seismic event that causes ground motions of 0.1g.

Figure #3 illustrates the 1% probability of exceedance in 50 years for the peak ground acceleration in the Libby area. This is equivalent to the 5000 year return period. Figure 3 shows that there is a 1% probability in 50 years that Flower Creek Dam will see a seismic event that causes ground motions of 0.2g. As mentioned above, preliminary studies indicate that ground motions of 0.2g can damage or fail the dam. *Thus, it can be concluded that there is a 1% probability in 50 years that Flower Creek Dam will experience a seismic event that will damage or fail the dam.*

For reference, the State of Utah standard is the 5000 year return period for high hazard dams and the 2500 year return period for significant hazard dams. Large federal dams are required to design their structures to withstand much larger quakes (10,000 year to 50,000 year return intervals).

The DNRC Dam Safety Program is in the process of adopting a seismic standard. For dams with significant downstream hazards (such as Flower Creek Dam), the 5000 year return period will likely be the standard. For dams with lower levels of risk, the 2500 year return period will apply. The Flower Creek Dam should be designed at a minimum to withstand an earthquake that generates a ground shaking of 0.2g (the 5000 year event). As mentioned above, recent engineering analysis questions the dam's ability to withstand a ground shaking of 0.2g. Thus it can be assumed with the knowledge we have to date that Flower Creek Dam does not meet the proposed Montana dam seismic standard.

Mike Stickney with the earthquake studies office conducted an independent assessment of the average return period of earthquakes in the Libby area according to magnitude. Mike concluded that that an earthquake with a magnitude of 5.0 might occur within 65 km of Libby once in a 49-year period and that a magnitude 6.0 earthquake might occur once in a 226-year period. Details of Mike's analysis and a discussion on the assumptions involved are contained in Appendix A.

Summary

- *A large earthquake (magnitude 7+) on nearby mapped faults will likely not generate high enough ground shaking at Flower Creek Dam to cause damage or failure. The likelihood of experiencing an earthquake of this magnitude on these nearby faults is extremely remote.*

- *A magnitude 6.5 earthquake on a blind fault in the vicinity of Libby could cause ground shaking sufficient to damage or fail Flower Creek dam. There is a 1% chance in 50 years that Flower Creek Dam will experience this level of ground shaking.*

- *Flower Creek Dam does not appear to meet the State's proposed seismic design standard.*

- *Based on recent earthquake records, it can be concluded that a magnitude 6.0 earthquake might occur within 65km of Libby once in a 226-year period.*

References

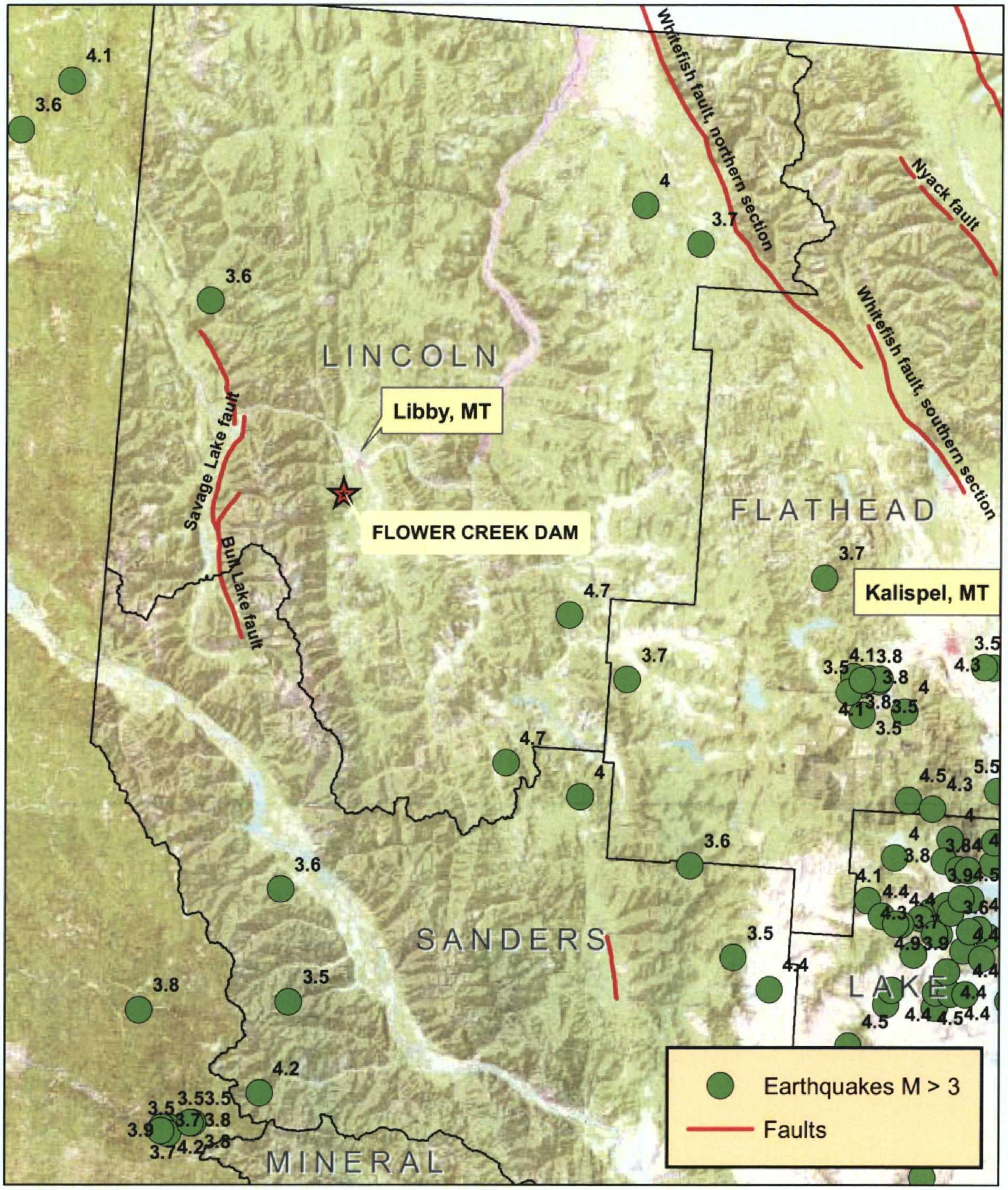
Wong, et al., 2005, Probabilistic Earthquake Hazard Maps for the State of Montana, MBMG Publication 117

NTL Engineering, 2010, Geotechnical Engineering Report, Phase II Geotechnical Investigation of Flower Creek Dam

Wong and Wright, (2008) MTShake Users Manual

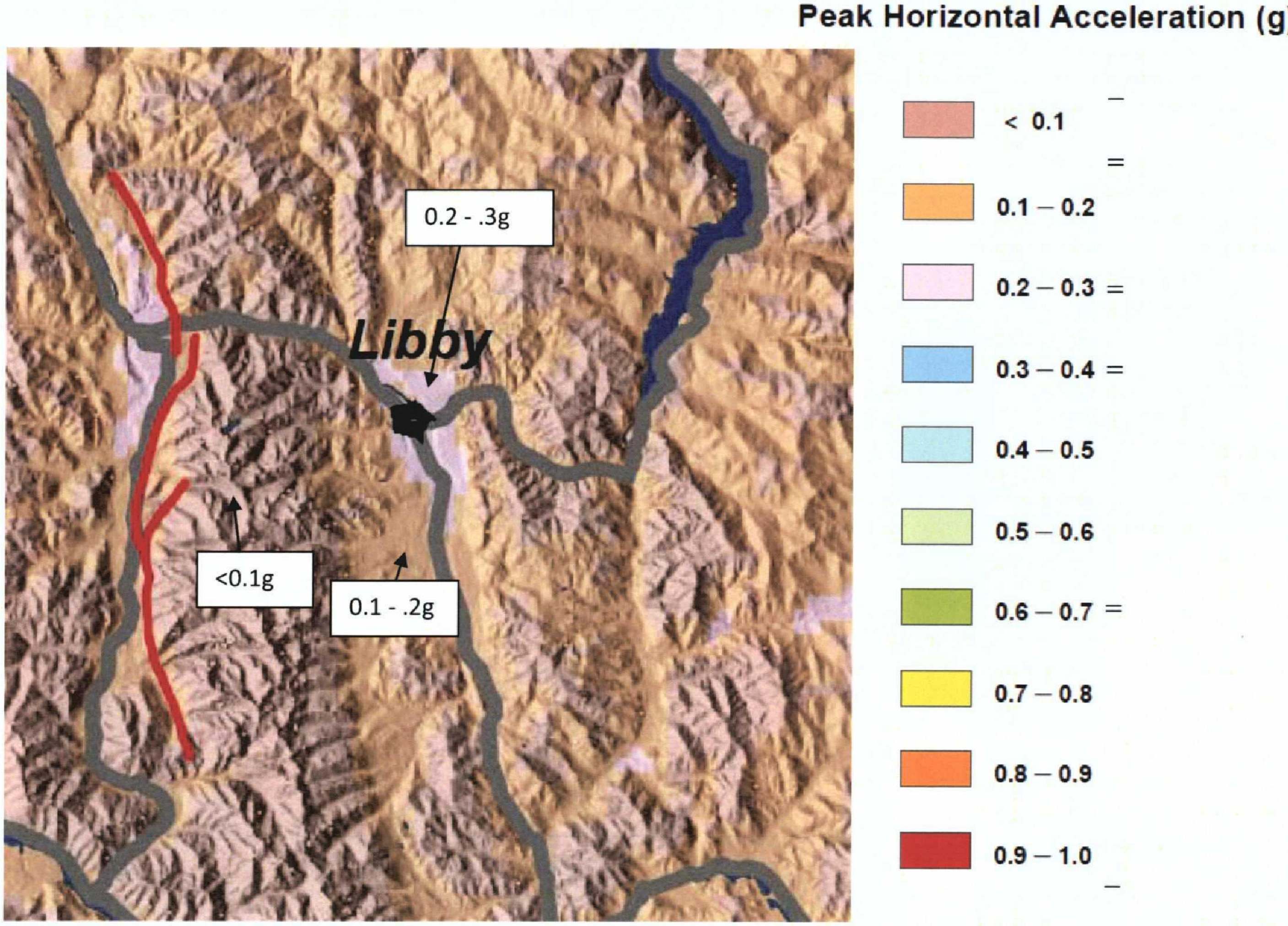
DNRC Earthquake Response Guidelines (2008)

Seismicity Near Flower Creek Dam



2% Probability of Exceedance in 50 Years
Peak Horizontal Acceleration (g) at the Ground Surface

~ 2500 year Return Interval



MBMG Publication XXX

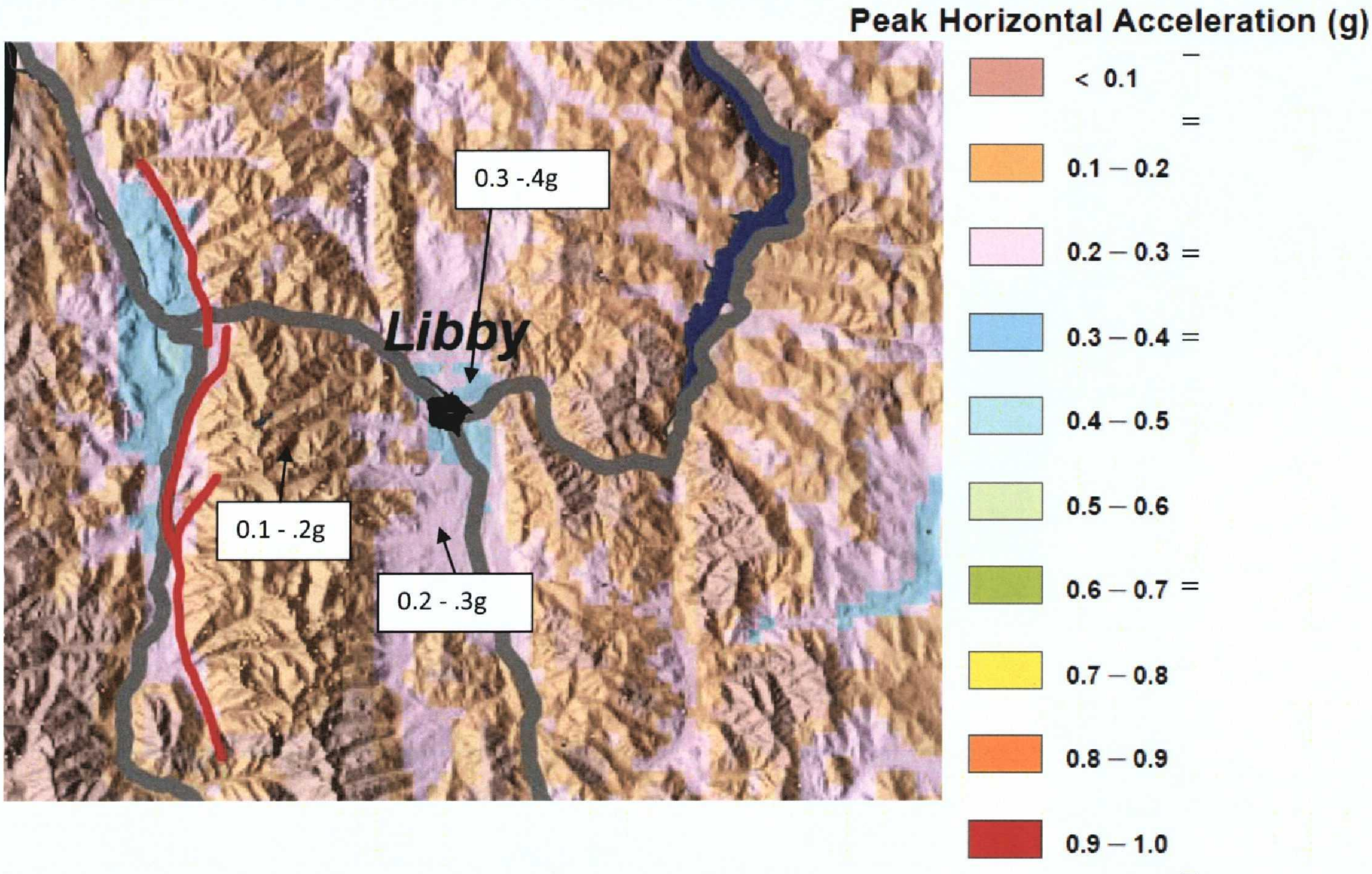
Probabilistic Earthquake Ground Shaking Maps for the State of Montana

by
Ivan Wong, Susan Olig, Mark Dober,
Douglas Wright, Eliza Nemser, David Lageson,
Walter Silva, Michael Stickney, Michele Lemieux
and Larry Anderson

2005

1% Probability of Exceedance in 50 Years
Peak Horizontal Acceleration (g) at the Ground Surface

~ 5000 year Return Interval



Appendix A

Technical Data and Discussions

Hello Ivan:

I have been getting quite a few inquiries about seismic hazards in NW Montana and their relationship to the Flower Creek Dam. I know that Michele has been receiving information from you on this topic and has been copying me with your replies.

I was surprised to hear that a M 7 earthquake only 18 km from the dam site would produce only 0.11 g. This must be at a specific period. The curves on the graphs you have sent certainly peak a values above 0.2 g. I presume that an concrete arch dam would have a fairly low natural period (several seconds?). Can you clarify what spectral frequency your values apply to. I guess the fundamental question is: would an M 7 earthquake on the Savage Lake fault generate more that 0.16 g in the frequency range of importance to the dam?

Thank you so much for any clarification.

-Mike

Michael Stickney, Director
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From:	Ivan_Wong@URSCorp.com
To:	Stickney, Mike
Cc:	Lemieux, Michele
Subject:	Re: PGA at Flower Creek Dam, Montana
Date:	Thursday, January 20, 2011 11:44:58 AM

Howdy Mike, hope you're doing well! Yup it is surprising that the PGA is that low but there are several factors going on. First the spectra were computed using the new NGA ground motion models which as you've heard, have lowered the ground motions for events of M > 7 particularly at short periods i.e., PGA. So if you look at the plot that I think Michele sent you, the average PGA is 0.11 g. PGA is defined at 0.01 sec period (100 Hz) so its the value on the far left. The site is reportedly hard rock so we have used a high Vs30 of 1500 m/sec. The NGA models are capped at this value. This Vs30 was a guess on our part. It could be lower but we don't have any data. The damsite is in the footwall in contrast to the hanging wall so most of the radiated energy if going away from the damsite. Finally all the NGA developers have normal faulting giving lower ground motions than strike-slip and reverse by about 20%.

I don't know what the natural period of Flower Creek Dam is but if its a concrete arch, then I would expect is rather stiff and its period is a few tenths of second. Don't know.

Hope this helps. Get back to me with more questions. Michele says she's hired one of dam engineers in Denver to help evaluate the dam. We'll know more. I'm sure I'll get roped into the evaluation.

ivan

Ivan G. Wong
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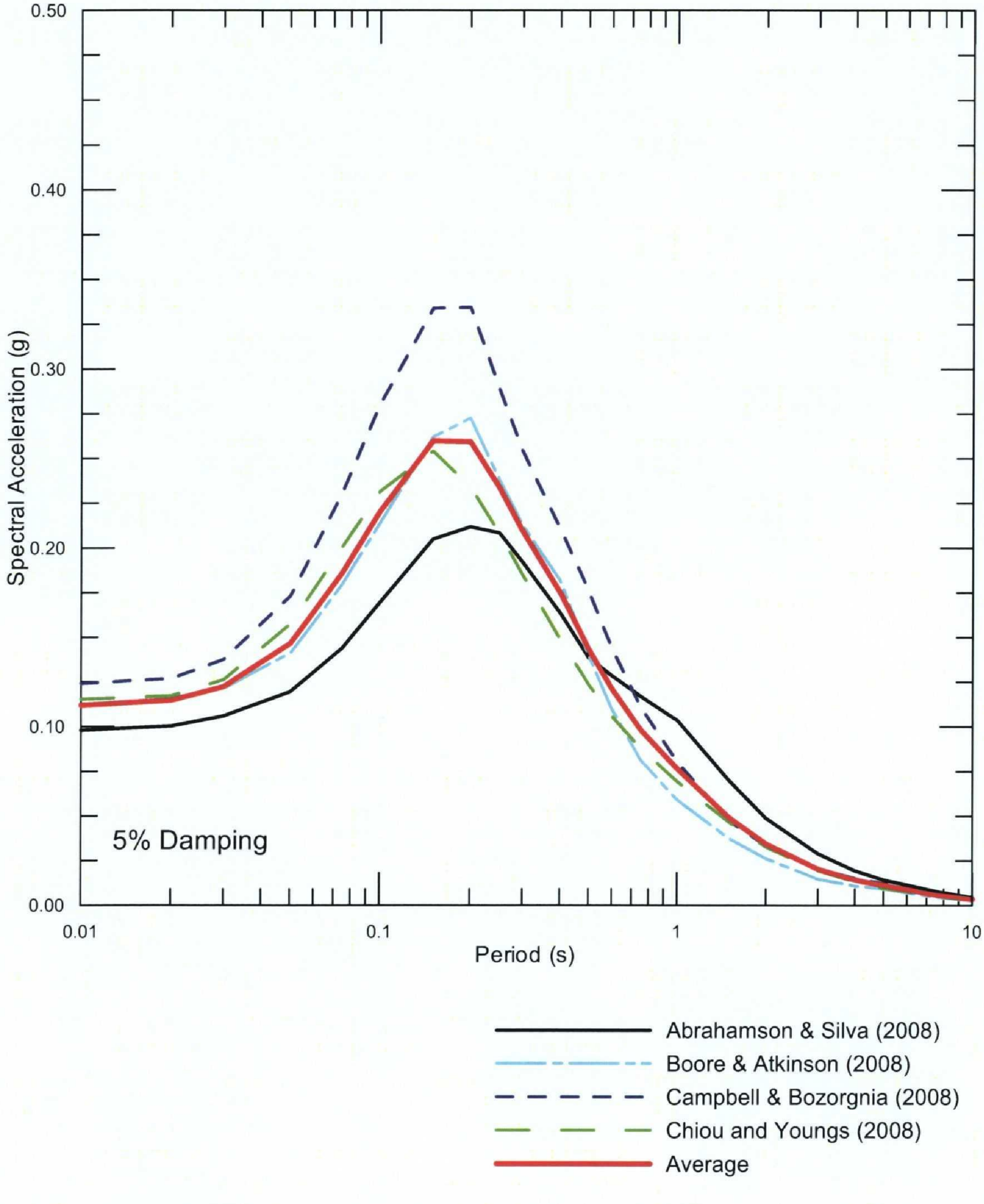
"Stickney, Mike" <MStickney@mtech.edu>

01/20/2011 09:20 AM

To "Ivan_Wong@URSCorp.com" <Ivan_Wong@URSCorp.com>
cc
Subject PGA at Flower Creek Dam, Montana



Mag = 7.0
Normal faulting
Dip = 55.0
R_{rup} = 18.0 km, R_{jb} = 18.0 km
In FootWall
V_S30 = 1500 m/s

URS	Project No. 00000000 Flower Creek Dam Montana	MEDIAN HORIZONTAL SPECTRA FOR THE SAVAGE LAKE FAULT	Figure #

From:	Stickney, Mike
To:	lcema@libby.org; Lemieux, Michele
Cc:	Deal, Edmond; Miller, Marvin
Subject:	Libby area earthquake recurrance information
Date:	Thursday, January 13, 2011 9:23:57 AM
Attachments:	Libby.pdf LibbyQks.txt

Hello Vic and Michele:

I went ahead and repeated the search of the Montana Bureau of Mines and Geology earthquake catalog for all earthquakes that we have located within a 65 km radius of Libby (taken as 48.388 N, 115.556 W) since January 1982. This search produced 163 earthquakes with magnitudes ranging from 0 to 4.7 (attached as LibbyQks.txt), the majority (87%) of which occurred since 2000 when seismic monitoring capabilities dramatically improved in northwestern Montana. I sorted these earthquakes according to magnitude and created a cumulative-number-of-earthquakes versus magnitude plot (attached as Libby.pdf). A linear regression of the larger magnitude earthquakes (M >= 2.6) is shown as a line on the plot and results in the equation: Log N = 3.07 – 0.66 M, where N is the number of earthquakes greater than or equal to magnitude M. The correlation coefficient for this regression is 0.992 (1.00 would be a perfect fit). I selected magnitude 2.6 as the minimum magnitude for this analysis because the data points for lower magnitude events begin to diverge systematically from a straight line, indicating incomplete detection and location of earthquakes with magnitudes less than 2.6 (particularly for the 1982-1999 time period).

Solving the above equation for various magnitudes and dividing by the 28-year catalog period gives the annual number of earthquakes of magnitude M. The inverse of the annual number of earthquakes is the average return period of an earthquake of magnitude M. These results are summarized in the following table.

Magnitude	Number / Year	Return Time (Years)
3.5	0.202	4.9
4.0	0.094	10.6
4.5	0.044	22.9
5.0	0.020	49.0
5.5	0.010	105.
6.0	0.0044	226.
6.5	0.0021	485.

The above results for earthquakes of magnitude 5.0 and larger extrapolates beyond both the time and magnitude range for which we have data and should be viewed with extreme caution. Furthermore, no attempt was made to remove dependent earthquakes, examples of which are aftershocks or swarm events that are clearly related to each other in time and space. This simplistic analysis assumes that the past 28 years of seismicity is characteristic of longer term earthquake behavior in the Libby region. These earthquake return times are similar to the concept of a 100-year flood. That is, knowing the size of the 100-year flood does not in any way help one predict when a flood that big will occur; only that, given enough historic data, that a flood of this size is likely to occur once in a 100-year period. With consideration of these limitations, this analysis suggests that an earthquake of magnitude S.0 might occur within 65 km of Libby once in a 49-year period, and that a magnitude 6.0 earthquake might occur once in a 226-year period.

I hope that this information may in some way assist your efforts. If you have any questions or need additional information, please do not hesitate to contact me.

-Mike

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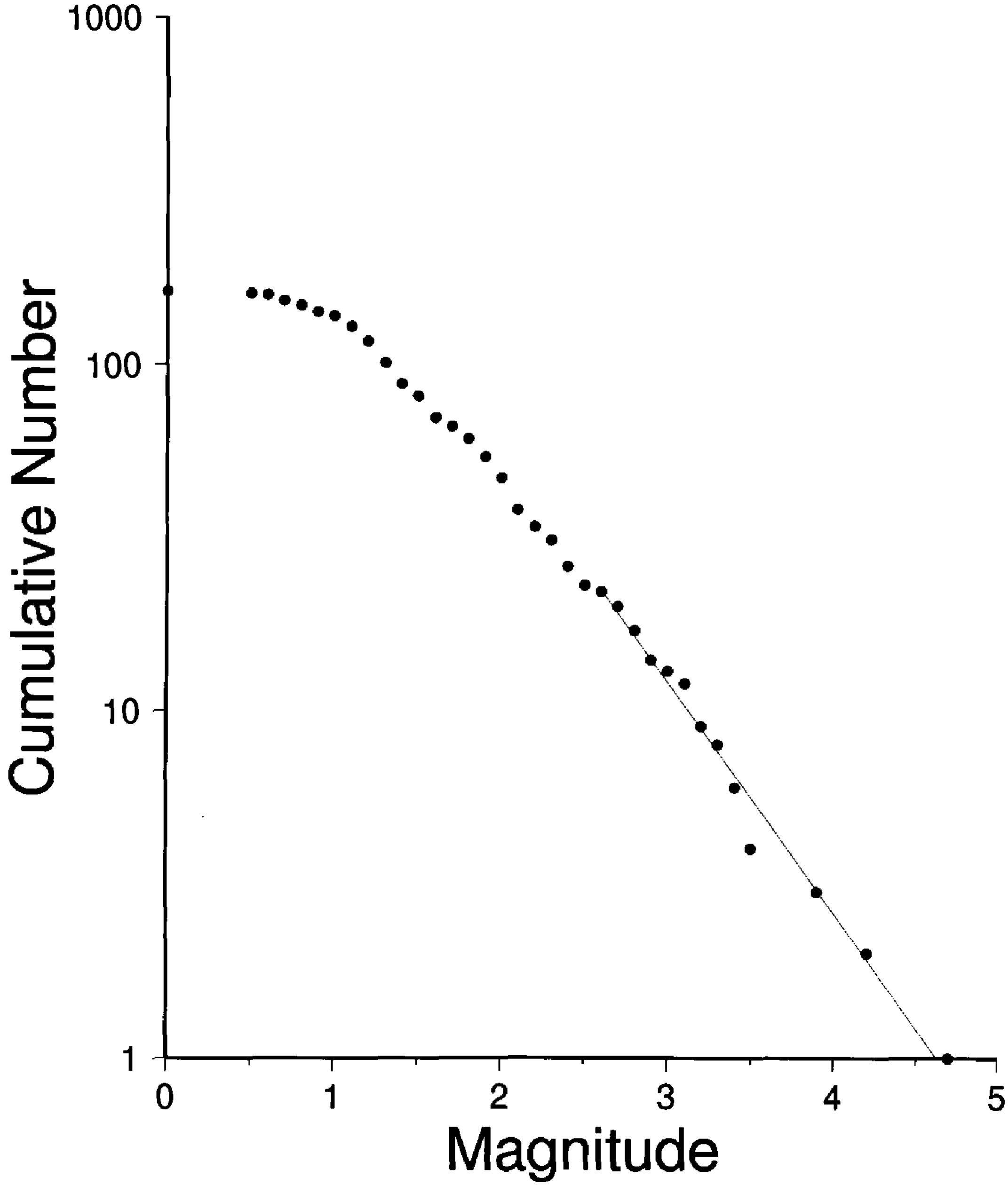

Cumulative Number
1000
100
10
1
0
1
2
3
4
5
Magnitude